DISCHARGE THROUGH TAINTER GATES

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Preface

The experimental work in connection with this thesis was done in the Hydraulic Laboratory of the University of Wisconsin, under the direction of Professor Charles I. Corp, to whom the authors are indebted for much valuable advice and assistance. The authors wish also to express their appreciation of the assistance rendered by Mr. Wm. Egan, laboratory assistant, who made observations of the upstream head on the dam during runs under submerged conditions.

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Introduction

The variation in stream flow of many American rivers is great, and in order to economically develop power on those streams the use of some type of movable dam is necessary. If a movable dam is used the headwater can be kept at a high level during times of ordinary flow, while during floods the channel can be opened and the pond level kept from rising much above normal. The purchase of flowage rights to large areas of land which would be damaged by floods held back by a fixed dam, or the loss of large amounts of power during periods of low flow, the two alternatives when a fixed dam is used, is unnecessary when a movable dam is used.

The tainter gate is a type of movable dam very widely used in connection with hydro-electric power developments. It is often known as the radial, pivoted, cylinder or drum gate, although the latter two names should properly be reserved for gates of another type. The gate consists of a sector of a cylinder, having its face covered with steel plates carried by radial frames pivoted at, or near, the center on the downstream side. The gate provides large openings in the dam and because of its shape and method of pivoting it is easier and quicker to operate than any other gate of equal size. This ease and simplicity of operation, together with its ability to withstand hard usage with little likelihood

of derangement of parts, makes the gate a desirable type to use where conditions require a movable dam.

The engineer, to correctly design a tainter gate dam to pass the maximum probable flood, must know something of the rate of discharge which can be expected through the gate opening when the gate is raised until its lower edge clears the upper surface of the water. When, as is frequently the case, the extent to which the gate can be raised is limited by details of the hoisting apparatus or by construction the lower edge of the wide-open gate may still be below the upstream pond level. It is, therefore, desirable to know how the interference of the gate will affect the flow.

Economical development of the water power resources of the country requires that records of stream flow be kept on all the principal rivers. Frequently the best place for measuring the discharge of a river is at the dam of an existing power plant. The quantity of water passing through the turbines can be determined quite accurately. However, the flow is very often more than can be utilized by the water wheels, and one or more gates may be partly opened to pass the excess water. It is then necessary to know the discharge characteristics of such gates for the given opening if the stream flow is to be correctly determined.

Irrigation project developments require the diversion of large quantities of water through canal headgates. These gates

for head works must usually be of the undershot type, and the loss of head through them must be small. It must also be possible to determine the flow through such gates with comparative ease and their discharge coefficient must therefore be known. While tainter gates have, at the present date, been used but little for irrigation canal headgates, the writers believe that there are many conditions under which their use is both feasible and desirable, and that their use in such works will greatly increase in the future.

Object of Experiments

The purpose of the writers in conducting these experiments has been to, (1) determine the value and variation of the coefficient of discharge of a spillway such as is often used for tainter gate dams, (2) to determine the value or values of the coefficient of discharge for the aperture formed by raising a tainter gate from its seat, (3) to determine the effect of submergence on the flow through the waterway provided by opening a tainter gate, and, (4) to determine the various factors which influence the discharge through tainter gates.

Historical Review

Thin-edged Weirs

Measurement of stream flow by observing the head on thin-edged weirs has been on a firmly established footing since the early experiments of James B. Francis made in 1852. These experiments were made to determine the exponent (n) in the weir formula, $Q = C\,L\,H^n$

in which Q = quantity of discharge in cubic feet per second

C = a coefficient, including the factors $\frac{2}{3} \cdot \sqrt{2g}$

L = the length of the weir in feet.

H = head on crest of weir measured to surface of still water in the upstream channel.

From these experiments Francis presents the final formula

$$Q = 3.33\,L\,H^{\frac{3}{2}}$$

In which, if there are end contractions the effective length

$L = L' - 0.1\,n\,H$, where

L' = measured length of weir

n = number of end contractions

and, if there is velocity of approach, the effective head is found from,

$$H^{\frac{3}{2}} = (D+h)^{\frac{3}{2}} - h^{\frac{3}{2}} \qquad \text{where}$$

D = measured or actual depth on the crest of the weir, usually determined as the difference of elevation of the weir crest and the water level, taken at a point sufficiently far upstream from the weir to avoid the surface curve.

h = Velocity head = $\frac{v^2}{2g}$ $\qquad$ v = average velocity of approach

The knowledge of the flow over sharp crested weirs was augmented by experiments made in 1877 by Fteley and Stearns on suppressed weirs, i.e., weirs with channel of approach equal in width to the length of the weir.

Beginning in 1886, Bazin, a French engineer, made a very complete set of weir experiments to determine the effect upon the discharge of:

(a) The velocity of approach

(b) The height of the weir

(c) The crest contraction

(d) The end contractions

(e) The form of the nappe

Bazin found experimental coefficients (m) to apply in the formula,

$$Q = m\, L\, D\, \sqrt{2g\, D}$$

for variations in each of the factors mentioned above. It should be noted that in the Bazin formula D can be corrected for velocity of approach in the same manner as for the Francis formula, and that (m) is then equivalent to $\frac{3.33}{2g}$ in the Francis formula. The results of Bazin's experiments are undoubtedly the most valuable data available today on the flow over weirs.

A number of weir formulas have been derived from subsequent analysis or recomputation of these earlier experiments considered in connection with later experiments made to check the coefficients determined. Among these later experiments may be mentioned

those of Hamilton Smith, and the Cornell University experiments for depths of flow on the crest exceeding those used by the early experimenters.

These many experiments have made possible the derivation of formulas and the determination of coefficients by the use of which the discharge over a standard thin edged weir can be accurately computed, within the limits of accuracy of head measurement.

In general, for precise measurements with standard thin-edged weirs--

1. The upstream crest should be sharp and smooth
2. The overflowing sheet should touch only the upstream crest corner
3. The nappe should be perfectly aerated
4. The upstream face of the weir should be vertical
5. The crest should be level from end to end
6. The measurements of head should show the true actual elevation of water surface above the level of the weir crest
7. The depth of leading channel should be sufficient to provide complete crest contractions, and, if they are not suppressed, the width of channel should be sufficient to provide complete end contractions.
8. A weir discharging from a quiet pond is to be preferred. If this is not available, the velocity of

approach in the leading channel should be rendered as uniform as possible and correction made therefor by the method employed by the experimenter in deriving the formula used. (W. S. P. #200, page 49.)

Broad-crested Dams and Irregular Weirs

Because of the many conditions which must be fulfilled the construction of a thin edged weir which shall be permanently useful to measure the flow of large streams is both difficult and expensive. It therefore becomes desirable to utilize existing dams to determine, at least approximately, the discharge of large rivers.

Several groups of experiments have been made to determine the proper coefficients for irregular weirs. The investigations of Bazin included experiments on various form of weir crest, including broad-crested weirs of several widths, with vertical upstream and downstream slopes, or with sloping aprons at various angles on the downstream or upstream sides, or on both. The United States Board of Engineers on Deep Waterways, in 1899, conducted a series of experiments at Cornell University to determine the flow over various irregular shaped dams. (Trans. Am. Soc. C. E., vol. 44). In 1903, a series of experiments were conducted for the U. S. Geological Survey by Gardner S. Williams, working under the supervision of Robert E. Horton. These experiments were made to determine the flow over weirs with broad crests (some with crests slightly in-

clined) and weirs with ogee sections. (W.S.P. #200, page 95.) Other experimenters have worked with one or two types of dams and have added to the available data on flow over irregularly shaped dams. While reliable coefficients have been determined for a few of the simpler types of dams, it may be said that, in general, the variation in the coefficient of discharge with variation in the shape of the crest is so great and irregular that no general rule of variation can be given, and the data and conditions of the original experiments must be consulted in each case.

Mr. Robert E. Horton, in Water Supply and Irrigation Paper No. 200 of the U. S. Geological Survey, has made available most of the existing data on flow over irregular shaped dams, and wherever possible has reduced the results obtained to coefficients for application in the simple formula

$$Q = C\ L\ H^{\frac{3}{2}}$$

so that the results of various experimenters are made comparable and can readily be applied to practical use.

<u>Submerged Dams</u>

If the water on the downstream side of a weir stands above the crest level, the dam is said to be submerged, drowned, incomplete, or partial. Dubuat, a Frenchman, published the first studies of the flow of water over submerged weirs in 1786. He used a suppressed weir and derived the theoretical formula that

is the basis of those used at present:

$$Q = c\frac{2}{3} L \sqrt{2g}\,(H-h)^{\frac{3}{2}} + c' L h \sqrt{2g\,(H-h)}$$

in which H and h are the heights of the water above the weir crest on the upstream and downstream sides respectively. Francis made a series of experiments on submerged weirs, the results of which were worked up in 1877 by Fteley and Stearns in connection with some experiments of their own. Francis made a later series of experiments in 1883, the results of which are published in his book "Lowell Hydraulic Experiments."

"Clemens Herschel in a paper before the American Society of Civil Engineers, 1885, declared himself dissatisfied with the submerged weir formula then in use, and presented a table of coefficients expressing the ratio of the head on the submerged weir to the head on a free fall weir of equivalent discharge for different percentages of submergence. These coefficients he derived from the experimental data obtained by Francis and Fteley and Stearns. Though he did not use the formula

$$Q = 3.33\, L\, (NH)^{\frac{3}{2}}$$

his coefficient N is always applied through this formula, which is therefore known as the Herschel Formula." (Clement, Conrad and Doolittle, U. of W. Thesis, 1915.)

Bazin made a very complete study of the flow of water over submerged weirs. For many of the model weirs of irregular section for which free discharge coefficients were obtained by him, he also obtained a duplicate series of coefficients with various

degrees of submergence. These coefficients were expressed as values of m for the formula:

$$Q = m\ L\ H\ \sqrt{2g\ H}$$

in which Q = discharge in cubic feet per second

m = coefficient

L = length of weir crest, feet

H = head on upstream side corrected for velocity of approach

g = acceleration due to gravity

Since each form of weir section requires a special formula or table of coefficients, little more can be done than to refer to the original data for each specific case.

A number of theses have been written at the University of Wisconsin on the discharge over submerged broad crested and ogee dams, both with and without approach slopes.

Large Rectangular Orifices

Very little data is available on the flow through large rectangular orifices. In the year 1827 M. M. Poncelet and Lesbros commenced at Metz an elaborate series of experiments upon the discharge through rectangular vertical orifices of various sizes, the largest being .656 feet square. Hamilton Smith, Jr., gives a summary of the results obtained in his book on Hydraulics. He also publishes a limited amount of data taken by himself in determining the discharge through long, narrow apertures cut through plank. The most complete

and reliable data concerning the flow of water through large orifices is given in a paper by Mr. Theo. G. Ellis, entitled "Description and Results of Hydraulic Experiments with Large Apertures at Holyoke, Massachusetts," and printed in the Trans. A. S. C. E., vol. 5, 1875. This paper presents the results of experiments on orifices two feet square , and two feet by one foot, discharging freely into air, and on the latter orifice submerged.

Investigations of the flow through submerged orifices have been conducted at the University of Wisconsin. C. B. Stewart made a series of experiments on orifices four feet square, and L. R. Balch investigated the flow through four-inch orifices. The results of these studies have been published in the University of Wisconsin Bulletins.

Tainter Gates

The writers have been unable to find any published record of experiments on the flow through the peculiar type of aperture formed by opening a tainter gate. Parker ("Control of Water") gives a very limited amount of data on the flow through "Orifices with Prolonged Boundaries", and on flow through "Sluices and Gates", under conditions which approximate more or less closely those existing at a tainter gate. Julian and Hinds, Engineers for the U. S. Reclamation Service, discuss the flow through canal headgates and present the results of

observations on two actual installations in the October, 1919, Reclamation Record. The data are very limited and leave much to be desired in this field.

Theory

Nomenclature

The significance of the letters used in the following discussion is as follows:

Q - quantity of discharge in cubic feet per second

A - area of gate opening in square feet

H - upstream head on dam, feet

H_s - downstream head on dam, feet

H_c - head on center of gate opening, corrected for velocity of approach

H_b - head on bottom of gate opening, corrected for velocity of approach

H_t - head on top of gate opening, corrected for velocity of approach

D_b - observed head on bottom of gate opening, feet

D_t - observed head on top of gate opening, feet

L - length of gate, feet

W - height of gate opening, feet

g - acceleration due to gravity, feet per second

h_v - head equivalent of velocity of approach

C, C_1, C_2, etc., - coefficients of discharge

h - difference in level between upstream and downstream channels for submerged orifice, corrected for velocity of approach

d - ditto, without correction for velocity of approach

N - Herschel coefficient $\frac{\text{Freefall head}}{\text{Submerged Head}}$ for same discharge

S - $\frac{\text{Downstream head on weir}}{\text{Upstream head on weir}}$

<u>Orifice</u>

It is not the purpose of this thesis to discuss the flow through standard sharp edged orifices or over standard weirs. Any standard text book on Hydraulics gives the derivation of the theoretical formulas to be used in computing the flow through such openings. This brief discussion is intended merely to show the relation between the flow through these standard structures to the discharge over the spillway of a tainter gate dam, and to the discharge through the peculiar type of orifice formed when a tainter gate is opened.

The ordinary formula for flow through a sharp edged orifice discharging freely into air, Fig. 1, is based on Torricelli's theorem, which states that "If a small orifice be opened in the side of a vessel containing water, the velocity of the escaping jet will be nearly equal to the velocity acquired by a body falling freely from rest through a vertical distance equal to the depth of the orifice below the free surface of the water," (Hoskins Textbook of Hydraulics) and is,

$$Q = CA\sqrt{2g\,H_c} \qquad (1)$$

$$= C\,L\,(H_b - H_t)\,\sqrt{2g\,H_c} \quad \text{for rectangular orifice}$$

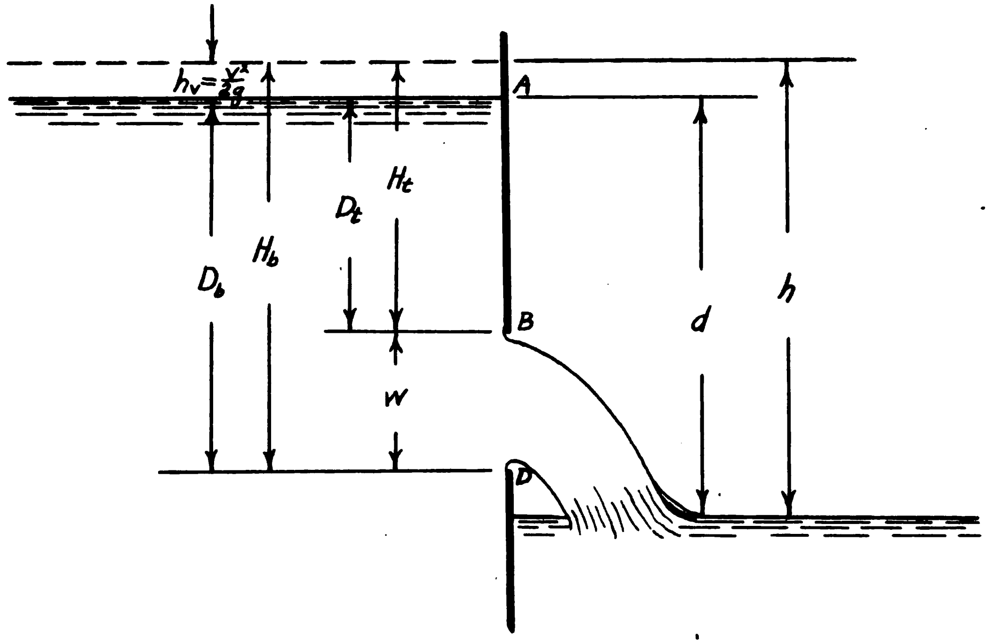


Fig. 1.

The coefficient C is introduced in the formula because the velocity of the water through the opening is not quite equal to the velocity which would be attained by a freely falling body falling a distance equal to the effective head, and because the area of the jet is considerably less than the area of the orifice, due to contraction on all edges. The coefficient C varies slightly with both the head and the dimensions of the orifice, but a good average value for sharp edged orifices is 0.61.

The head H_c is usually taken to the center of the orifice if the size of the opening is small in comparison to the head acting. If that condition does not exist, the variation in effective head at different points in the orifice must be con-

sidered and the formula becomes, for a rectangular orifice of width L and depth $H_b - H_t = W$:

$$Q = C_1 \frac{2}{3} \sqrt{2g}\ L\ (H_b^{\frac{3}{2}} - H_t^{\frac{3}{2}}) \qquad (2)$$

H_b = effective head on bottom edge of orifice = $D_b + h_v$

H_t = effective head on top edge of orifice = $D_t + h_v$

If the orifice instead of being in a vessel is in a bulkhead across a channel, and if the area of the orifice is a considerable part of the cross sectional area of the channel, the water will approach the orifice with an appreciable velocity, V. The fall of the free surface which must have been expended in producing velocity V is, then, $h_v = \frac{V^2}{2g}$. To get the effective head the measured heads must therefore be corrected by the quantity h_v.

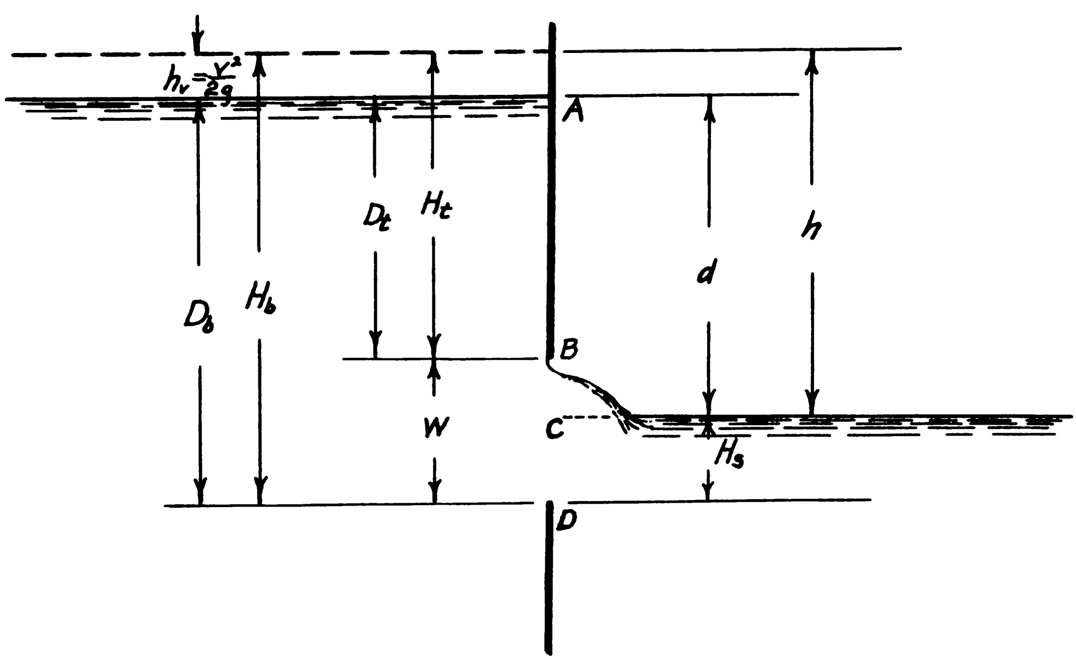


Fig. 2

If the orifice does not discharge freely into air but is partially submerged (see Fig. 2) the conditions are modified somewhat. By what precedes, the discharge Q_1 through BC, the portion of the orifice clear above the tail-race, is

$$Q_1 = C_1 \frac{2}{3} \sqrt{2g}\ L\ (h^{\frac{3}{2}} - H_t^{\frac{3}{2}})$$

The flow through the submerged portion CD is evidently under the effective head h and the discharge is therefore given by

$$Q_2 = C_2\ L\ (H_b - h)\ \sqrt{2g\ h}$$

Assuming that $C_1 = C_2$ the total discharge is

$$Q = Q_1 + Q_2 = C_2 \sqrt{2g}\ L\ \left[\frac{2}{3}(h^{\frac{3}{2}} - H_t^{\frac{3}{2}}) + (H_b - h)\sqrt{h}\right] \qquad (3)$$

If the orifice be wholly submerged (Fig. 3) the effective head on all parts is evidently h and the discharge equals,

$$Q = C_3\ \sqrt{2g}\ L\ (H_b - H_t)\sqrt{h} \qquad (4)$$

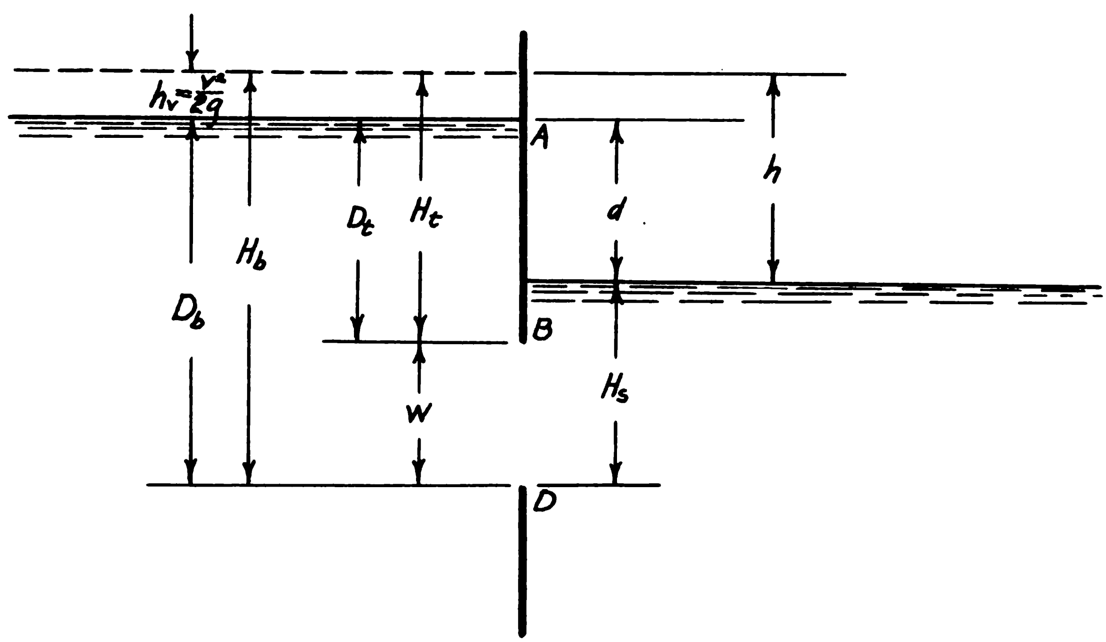


Fig. 3

The area of the jet issuing from the orifice in each of the above cases will be less than the area of the orifice. The coefficient C will vary with the ratio, area of contracted section to area of orifice, and will have its least value for a sharp edged orifice in which complete contraction takes place. The borders of the channel and the edges of the orifice must at no place be less than three times the least dimension of the orifice apart if complete contraction is to occur. (Parker, page 146.) Since the difference between the area of the contracted section of the jet and the area of the orifice is caused by the convergence of the streams of water approaching the edge of the orifice from the interior of the resevoir in which the orifice is made, any border or thickening of the edge of the orifice will partially prevent this convergence, and will consequently increase the value of C, the coefficient of discharge. (Parker, page 147.)

Weir

If the water level in the resevoir supplying an orifice becomes so low that the free surface just passes the upper edge of the orifice, the latter ceases to act as an orifice and then acts as a weir. Under those conditions the upper edge of the orifice could be removed without affecting the flow.

The discharge over a sharp-edged weir may be found by putting D_t, in equation (2) above, equal to 0. Thus equation (2) becomes:

$$Q = \frac{2}{3} C L \sqrt{2g} \left(H_b^{\frac{3}{2}} - h_v^{\frac{3}{2}}\right) \qquad (5)$$

and if the area of the approach channel is large in comparison to the area of flow over the weir, the velocity of approach is small and h_v may be disregarded, giving the general weir formula as generally used:

$$Q = \frac{2}{3} C L \sqrt{2g} \; H^{\frac{3}{2}} \qquad (6)$$

where H is the depth of the crest of the weir below the surface of still water above the weir.

The effective sectional area of the water flowing through a rectangular weir is less than L H because of (a) crest contraction, (b) end contraction, and (c) the fall of the free surface towards the point of discharge. To correct the theoretical discharge for the effect of these contractions of the stream the coefficient C is introduced in the formula. It is reasonable to assume that the diminution of the actual sectional area, L H, due to crest contraction and the fall of the free-level surface is proportional to the length L of the opening, and that weirs with the same shaped crest will have the same coefficient of discharge regardless of the length, providing the effect of end contractions is eliminated. Francis (Lowell Hydraulic Experiments) finds that the end contractions are proportional to the head on the weir and from a series of experi-

ments presents the final formula:

$$Q = 3.33\ (L - 0.1\ N\ H)\ H^{\frac{3}{2}} \qquad (7)$$

L = measured length of weir in feet

N = number of end contractions

Q = quantity of discharge in feet

H = head on weir crest in feet

If the weir is of considerable length compared to the depth of flow the effect of end contractions is small and may be neglected, and the formula reduces to:

$$Q = C\ L\ H^{\frac{3}{2}} \qquad (8)$$

which is a most convenient form to use, since the effects of all variables are included in one coefficient, which is then a direct index of the flow over one weir as compared with another.

Theoretical discussions of the flow of water over submerged weirs usually consider the discharge as consisting of the flow over a weir under a head $(H - H_s)$ (See Fig. 4), and of the flow through a submerged orifice of depth H_s under a head $(H - H_s)$. The formula for the total discharge is then:

$$Q = C_1 \sqrt{2g}\ L \left[\frac{2}{3} (H - H_s) + H_s (H - H_s)^{\frac{1}{2}} \right] \qquad (9)$$

This formula reduces to a simple form and is more comparable to the free fall weir formula if it is expressed in terms of the upstream head on the weir, H, and the ratio of submergence of the weir or $\frac{H_s}{H} = S$. The formula then becomes:

$$Q - C\ L\ H^{\frac{3}{2}}\ (1 - S)^{\frac{1}{2}}(1 + \frac{S}{2}) \qquad (10)$$

in which $C = C_1\ \frac{2}{3}\sqrt{2g}$. The produce of the last two terms in this formula represents the theoretical ratio of the discharge over a submerged weir to that over a freefall weir under the same upstream head, and is always less than unity. A simple computation shows that the values of this ratio decrease slowly for submergences up to 50%, but decrease very rapidly at high submergences.

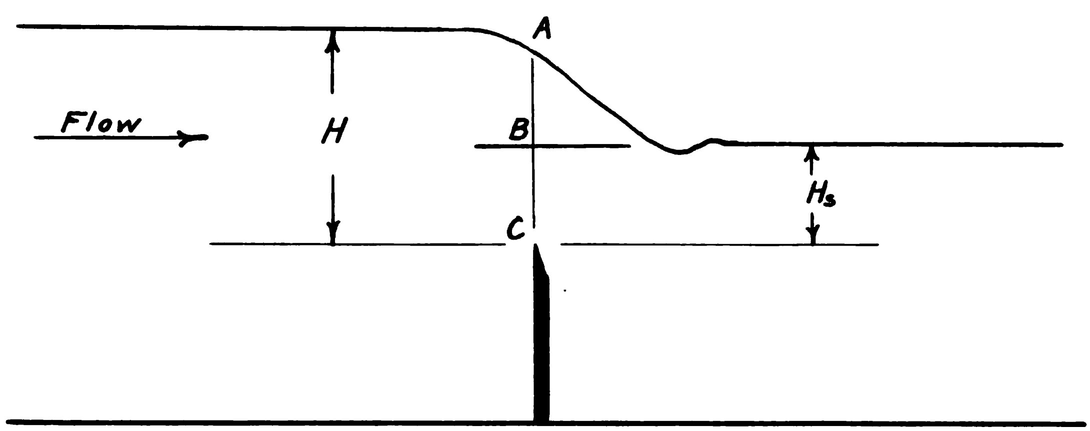


Fig. 4.

The Herschel method of attacking the problem of the flow over a submerged weir consists of reducing the measured head to an equivalent head which would give the same discharge over a free overflow. Herschel determined the value of the multi-

plier N to be applied to the head in the formula:

$$Q = C\,L\,(N\,H)^{\frac{3}{2}} \qquad (11)$$

in which,

Q = discharge of a submerged weir

C = coefficient of discharge of the weir under freefall conditions

L = length of weir

H = measured difference between upstream and downstream heads on a submerged weir

The discharge over a submerged weir, according to Herschel's formula, bears the ratio $N^{\frac{3}{2}}$ to that over an unsubmerged weir under the same head.

The formulas given above are applicable to sharp-edged openings, an edge being considered sharp when the jet springs clear at the inner face of the opening and does not again come in contact with the edges. Since the effect of the contraction is cared for in the formulas by the use of the coefficient C, the same formulas may be used for other than sharp edged openings if the proper value of C is used to care for the effect on the area of the issuing stream of any suppression or increase in the contraction and to care for any change in the frictional resistance to free passage of the water through the opening.

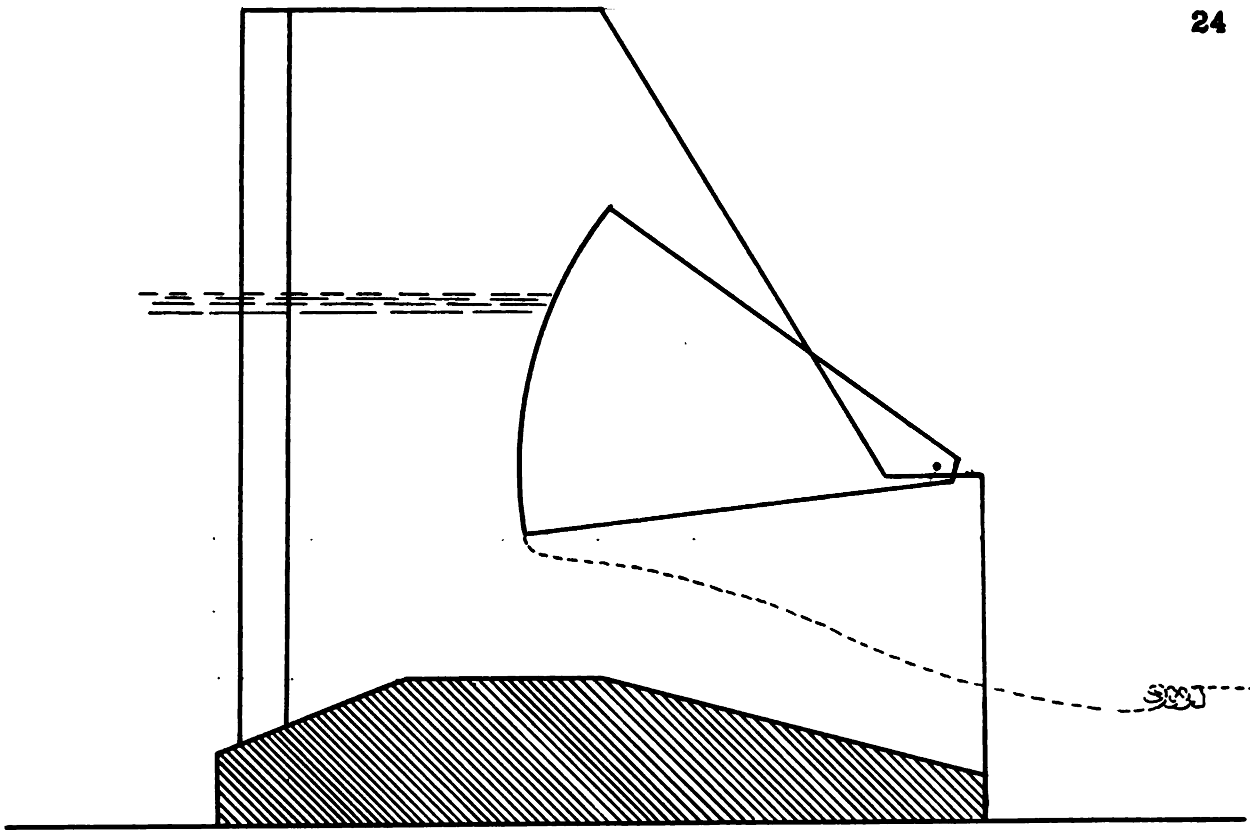


Fig. 5.

Experimental Dam

The peculiar type of weir and orifice under consideration in this thesis is shown in Fig. 5 and Plate 4. The effect of an approach slope such as AB has been found by many experimenters (See experiments of Bazin and U. S. Deep Waterways Board, Water Supply Paper #200, and 1916 Thesis of Conrad, Fowler and Parker) to be a reduction of the bottom contraction and a consequent increase in discharge. The effect of a flat crest such as BC on the discharge is found to vary with the ratio of the breadth of the flat crest, to H, the head on the weir. The ratio of C, the coefficient for a thin-edged weir, to C', the coefficient for a broad crested weir is found by Bazin to con-

tinually increase as H increases, B remaining constant, and to continually decrease as B increases, H remaining constant. Similar results were obtained by Mullins. (W.S.P. #200, page 118) The effect of a downstream apron such as CB (Fig. 5) is to slightly decrease the discharge since it supports the jet above its free trajectory and tends to produce a back pressure at the crest. The piers at the ends of the experimental weir evidently would partially suppress end contractions and should theoretically increase the discharge.

Description of Apparatus

Introductory

The general plan of apparatus used in the experimenting conducted for this thesis is shown on Plate II. All of the apparatus used, with the exception of the dam and gate shown on Plate IV, was already at the Hydraulic laboratory, and was merely set up to suit the purpose. Various drawings and pictures are shown throughout so as to enable the reader to obtain a clearer conception of the apparatus employed.

Source of Water Supply

The water used in these experiments was obtained from Lake Mendota, upon whose shore the laboratory is located. The

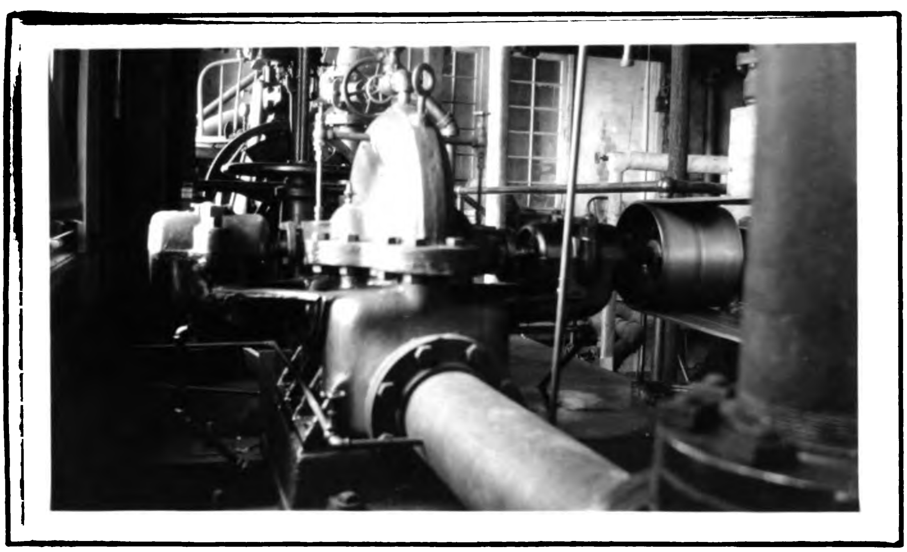


Fig. 6.

water was pumped by a six inch centrifugal pump into the large concrete reservoir which is located on the hill behind the

Engineering Building. From here it was drawn as needed. Some of the larger gate openings required more water than could be furnished continually by the reservoir so it became necessary to operate the pump at the same time.

Pipe Lines

As shown on Plate I the piping system was so arranged that water could be drawn either from the pump, reservoir, or from both. A one inch feed pipe from the University pressure mains is also shown. This was used to obtain a finer degree of regulation of the water going over the measuring weir.

The Weir Box

The discharge into the experimental channel, for all the smaller runs, was measured by a sharp crested, rectangular, contracted wier two feet long set in a weir box over the head of the channel.

The construction of this box is shown in detail in Plate II. It was built of two inch cypress planks tongued and grooved. The inside length was seventeen feet, three inches; the width, four feet, three inches; and the depth four feet, three and a half inches. The box was held together by sets of four-by-four inch timbers bolted together with half inch tie rods.

It was supported over the head of the channel on a concrete parapet wall on one side and by four-by-four inch blocks

on the other side. An iron jack on the block under the front timber was used to level up the weir crest after the box was in position.

A two and a half inch gate valve was placed in the side of the box two inches above the bottom near the rear to be used for draining the box and in setting the zero of the hook gage.

The Measuring Weir

The measuring weir set in the end of the box was a rectangular notch two feet wide and one and a half feet deep.

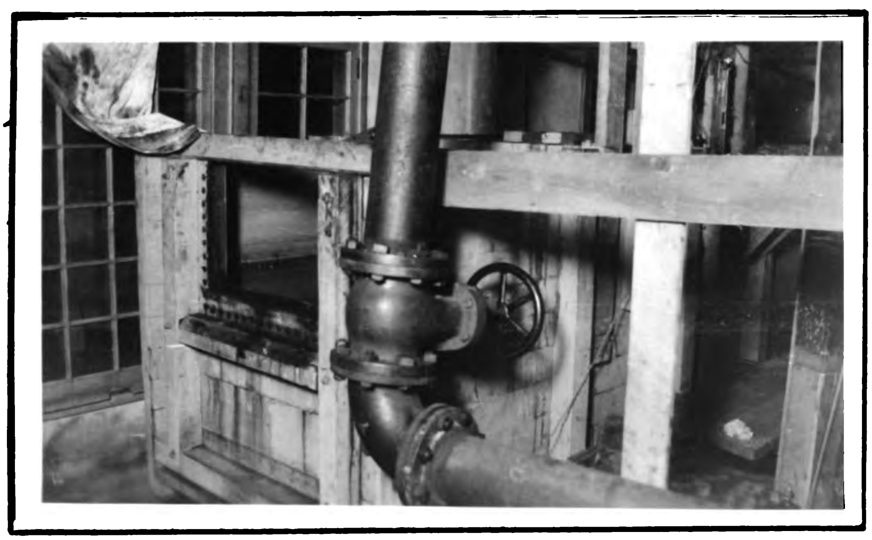


Fig. 7.

It was cut from steel plate three-sixteenths of an inch thick, the edges of the downstream side being beveled, leaving a crest

one-sixteenth of an inch wide. The plate was constructed so that when the crest was level the sides were vertical. It is shown in Fig. 7.

Weir Box Baffles

As shown on Plate II a system of baffles was arranged in the weir box insuring a steady flow of water. These baffles began three feet from the rear end of the weir box. The first set consisted of two solid partitions of three-quarter inch boards, the first extending upward from the bottom of the box and the second, eighteen inches downstream, extending downward from the top of the box and over-lapping the first by a little over one foot. Ten inches further downstream was the first of the second set of baffles. These baffles were slat baffles placed edgewise in the current. The upstream baffle slats were vertical and the downstream baffle slats were horizontal.

The Venturi Meter

With large gate openings the amount of water needed exceeded that which the weir could supply, so it became necessary to set up the venturi meter shown on Plate III.

This venturi meter was set up to the left of the downstream end of the measuring weir box and discharged into the experimental channel at the same place as the measuring weir. A mercury pressure gage was employed to measure the difference in pressure at the mouth and throat.

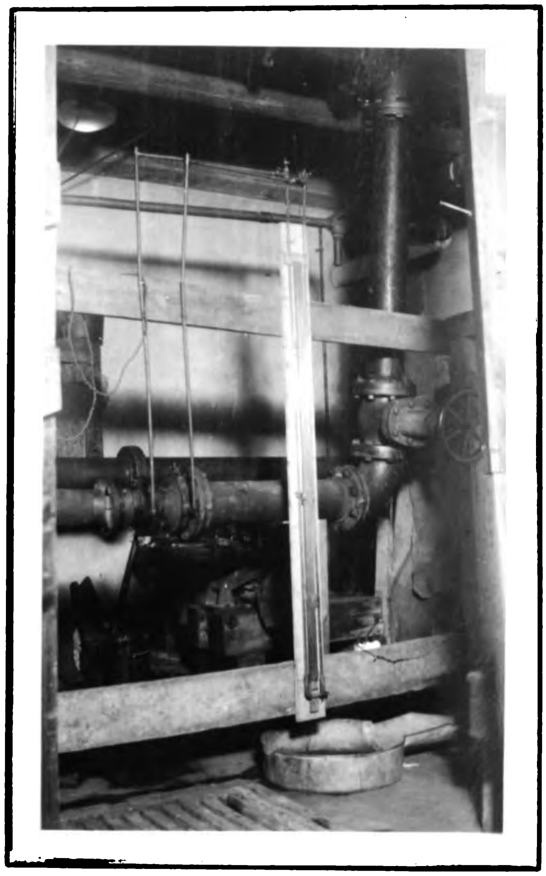


Fig. 8.

The Experimental Channel

This channel is along the south wall of the laboratory on the experimental floor. It is ninety-five feet long, four feet wide, and four and a half feet deep. The south side is lined with one inch matched sheeting six inches wide nailed to timbers set flush with the face of the concrete. On the north side the lining is nailed to two-by-fours attached edgewise to the permanent timbers in the concrete wall. Upstream of the

experimental dam no attempt was made to keep water from going through the lining but the cross wall of the dam extended back to the concrete walls so that no water could go below the dam without passing through the gate opening.

A system of baffles, similar to those used in the weir box, consisting of four solid board baffles and two slat baffles was built in the head of the channel to quiet the turbulent flow.

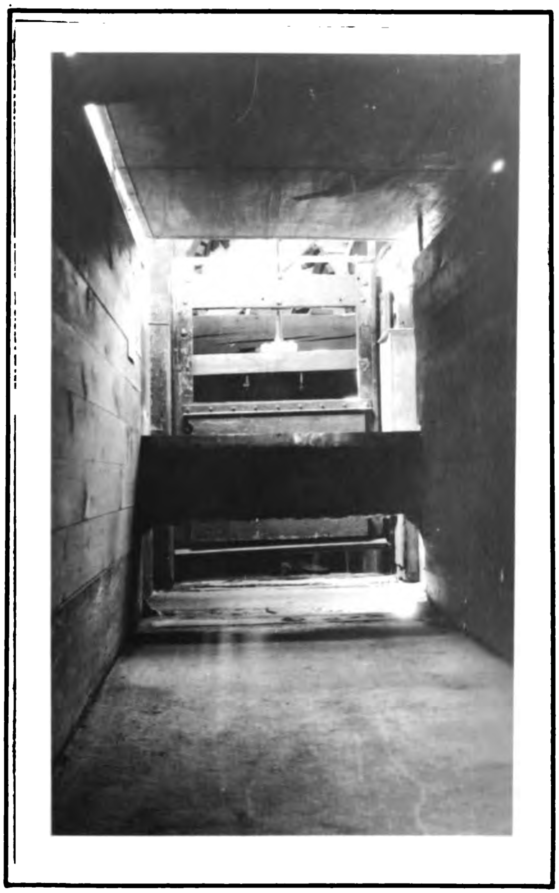


Fig. 11.

Fig. 9.

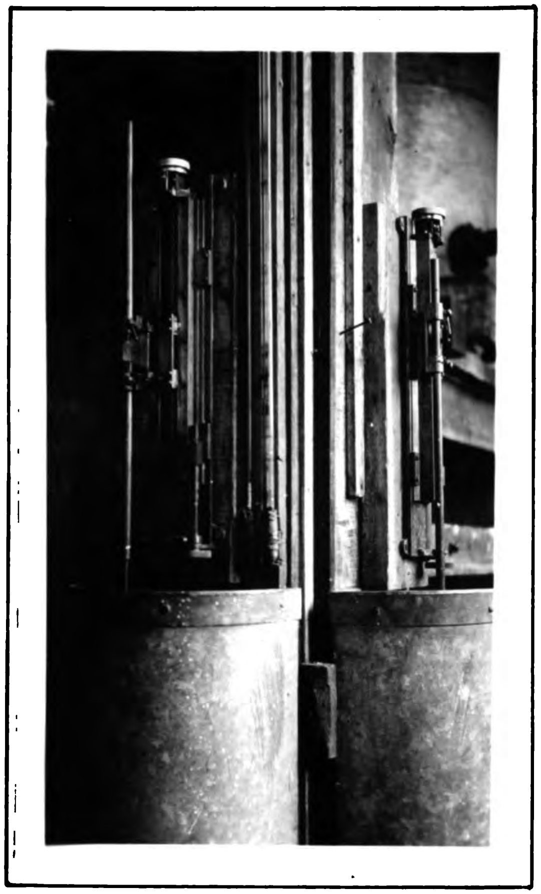
Fig. 10.

The Experimental Dam and Tainter Gate

This apparatus is shown on Plate IV, and in Fig. 9 and 10.

The dam was built of concrete and was located forty-six feet from the head of the channel. The crest was six inches above the bottom of the channel, eight inches wide, and level. From the upstream edge of the crest, the approach sloped down with a three to one slope for eight inches, and then dropped vertically to the bottom of the channel. From the downstream edge of the crest, the apron sloped down with a four to one slope for sixteen inches, and then dropped vertically to the channel bottom. The width between the two vertical walls was twenty inches. These walls were four inches thick with the upstream edge pointed to a right angle.

The tainter gate was built of wood with a galvanized iron face. This face was curved in the shape of an arc with a seventeen and a half inch radius. It was held in place by a half inch iron rod passing through the back of the gate and in bearing on timber blocks placed in the back of the vertical walls. The raising and lowering of the gate was controlled by two strips of wood fastened to the frame of the gate by bolts and extending up to a framework of two-by-fours where they were held at the desired place by hand clamps. A strip of rubber packing was attached to the vertical sides of the gate to prevent water from leaking through.



The Control of Submergence

The control of the degree of submergence was effected by means of a vertical lift gate located forty-nine feet downstream from the dam. This gate was not water-tight, but obstructed the flow of water enough so that the desired submergence was obtained. It was operated by a hand wheel placed at the center of the gate. A view of the gate is shown in Fig. 11.

Hook Gages

The measurements of head at all places, except the venturi meter were made with hook gages. For measuring the head on the measuring weir the special type designed by Prof. D. W. Mead was used. The hook, fastened on the lower end of a brass tube, was moved by a micrometer head screw. The principal scale, one foot long, was graduated in tenths and hundredths of a foot. The pitch of the screw was one hundredth of a foot, and the circumference of the head was divided into one

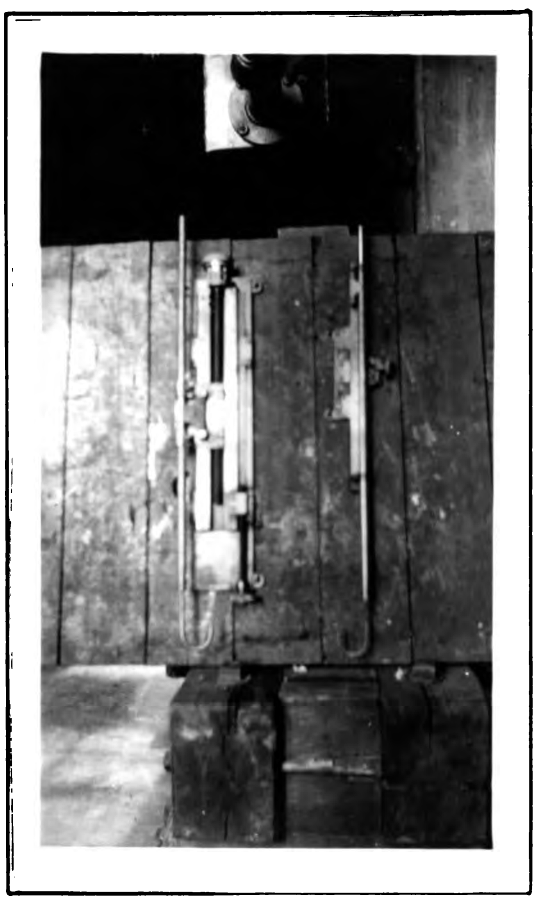


Fig. 12.

hundred equal parts so the measurements could be read to ten-thousandths of a foot. To measure heads beyond the range of the scale the brass rod was loosened and moved five-tenths of a foot, the tube having divisions on it for this purpose.

The other type of hook gage was not as delicate an instrument in that it read only to a thousandth of a foot. Its scale was divided into tenths and hundredths of feet and by means of a vernier was read to thousandths of a foot. This type was used for measuring the upstream and downstream head at the experimental dam. Both types of gages are shown in Fig. 12.

Piezometer Openings

The connection from the weir box to still basin No. 1 was made by a short piece of one inch pipe, the end being set flush with the inside of the weir box, six and a half feet back from and seven and a half inches below the crest of the measuring weir.

The upstream head was measured six feet upstream from the dam and four inches above the bottom of the channel by the use of the special piezometer shown on Plate V. The connection between still basin No. 2 and the piezometer was made by a one inch pipe and a flexible rubber hose.

The downstream head was measured in still basin No. 3, which was located eight feet below the experimental dam. Water was admitted to this still basin through four one-eighth inch holes punched in each side of the still basin one half inch above the bottom.

Still Basins

The use of still basins was necessary in order that the various heads could be read accurately.

The still basin for Gage No. 1 was hung outside of the weir box six and a half feet back from thecrest of the measuring weir. It is shown in place on Plate 2.

The still basin for Gage No. 2, measuring the upstream head on the experimental dam, was located nine feet downstream from the experimental dam. It is shown in Fig. 13

Fig. 13.

Still basin for Gage No. 3, measuring the downstream head, was located eight feet downstream from the experimental dam in the center of the channel. It is shown in Fig. 9. It was of a special shape, being long and narrow, pointed in the front and rounded in the back; so as to interfere with the flow of water as little as possible. The arrangement for supporting the basin was constructed so that the basin could be raised or lowered with changing level of the water.

Methods of Experimenting

Determination of the Zero of the Hook Gages

The zero of Gage No. 1 on the measuring weir was obtained by reading the gage when the water surface in the weir box was at the level of the crest. This position of the surface was determined by passing an electric light across the crest, the image of the filament remaining undistorted when the surface was at the crest level, and showing a distortion when it was above or below.

The zero of Gage No. 2, used to measure the upstream head on the experimental dam, was determined in two ways. The first method employed was to bring the water up to the level of the crest of the dam and read the gage. A carpenter's level was set on the crest of the dam and let extend out over the water surface as a means of determining when the level had been reached. The second method employed was that of using an engineer's level and a leveling rod. The rod reading was taken for several points across the crest and these values averaged. The rod reading was then taken on the point of the hook gage. The zero could then be computed.

For getting the zero of Gage No. 3, two methods were also employed. The first was by the use of the engineer's level and leveling rod, and the second by the use of a syphon between Gages 2 and 3. The holes in the still basins were corked tight, and the water allowed to syphon over through

a rubber hose. When the level of the water in both still basins was the same the hook gages were read simultaneously. It was then a simple matter to compute the zero of the hook gage.

Several observations were taken for getting the zero of all the hook gages serving to obtain both a check and an average value. The values obtained for Gage No. 2 showed a range of four thousandths of a foot and those obtained for Gage No. 3 a range of one thousandth of a foot.

Reading the Hook Gages

Hook gage readings were taken at half minute intervals during the runs. Where the water surface oscillated the readings were taken at the troughs and crests of the waves. The effect of lost motion in the screw was eliminated from the average readings by always lowering the point of the hook below the water surface and bringing it up to the surface.

Reading the Mercury Pressure Gage

Considerable oscillation occurred in the mercury pressure gage and the average was obtained here also by reading the high and low points.

Measuring the Gate Opening

The method used for measuring the gate opening was by direct measurement with calipers and scale. After the gate had been set at the approximate desired opening, readings

were taken at four inch intervals across the width of the crest. These values were averaged and the average recorded as the gate opening.

The Runs

Runs were made under four different conditions. The first runs made were with the gate removed, unsubmerged and with various upstream heads. The gate was left out for the second series also, but in this case the upstream head was kept approximately constant for the entire series, while the degree of submergence was varied. The gate was placed in position for the third series of runs. For each of the several gate openings a number of runs were made with different values of the upstream head, the head varying from a level just above the lower edge of the gate to the level used in the constant head runs above mentioned. The discharge for this series was free. A fourth series of runs was made with the upstream head as nearly constant as it was possible to maintain it, while for each of the several gate openings, the downstream head was varied from below the crest of the spillway to a level but little below that in the upstream channel. The gate openings used in this last series were made as near the same as those used in the free discharge runs as was possible with the means used in setting the gate. In runs where there was no submergence, the heads on the measuring weir and the upstream head on the dam were read and recorded. When submergence existed the downstream head on the dam was also recorded.

Computations

The arithmetical average of all readings taken during a run were used in computing the corrected heads, and the coefficients of discharge. Heads have been corrected for velocity of approach by assuming that the correction to be added to the upstream head was equal to the square of the average velocity through the entire channel section at the upstream gage divided by twice the acceleration due to gravity, or $h_v = \frac{v^2}{2g}$. This is approximately correct for low velocities. Theoretically, under submerged conditions, the head due to the velocity of retreat should be subtracted from the depth of submergence, but no such correction has been made in these computations.

For convenience in converting the head on the measuring weir into quantity of discharge, the table of logarithms of heads and corresponding discharges (see Appendix B) was prepared. Use of this table made it possible to determine to thousandths of a cubic foot the discharge corresponding to a given head. As the venturi meter was only used for a very few runs no such table was prepared but the logarithm of discharge was read directly from a large scale curve similar to the logarithmic head-discharge curve shown in Appendix C.

With the exceptions hereafter noted, six-place logarithm tables were used in the computations. Since the logarithm of discharge could only be determined to four places

with accuracy, it was considered sufficiently accurate to use logarithms to the nearest four places, and that was the methed performed. The area of channel of approach and the corresponding velocity were computed with a 10-inch slide rule. Values of the velocity head were taken from Table 1, Water Supply and Irrigation Paper No. 200. Three-halves powers of heads were taken from tables 5 and 6 in the same paper. Percent submergence was computed with the slide rule.

GRAPHICAL SUMMARY OF RESULTS

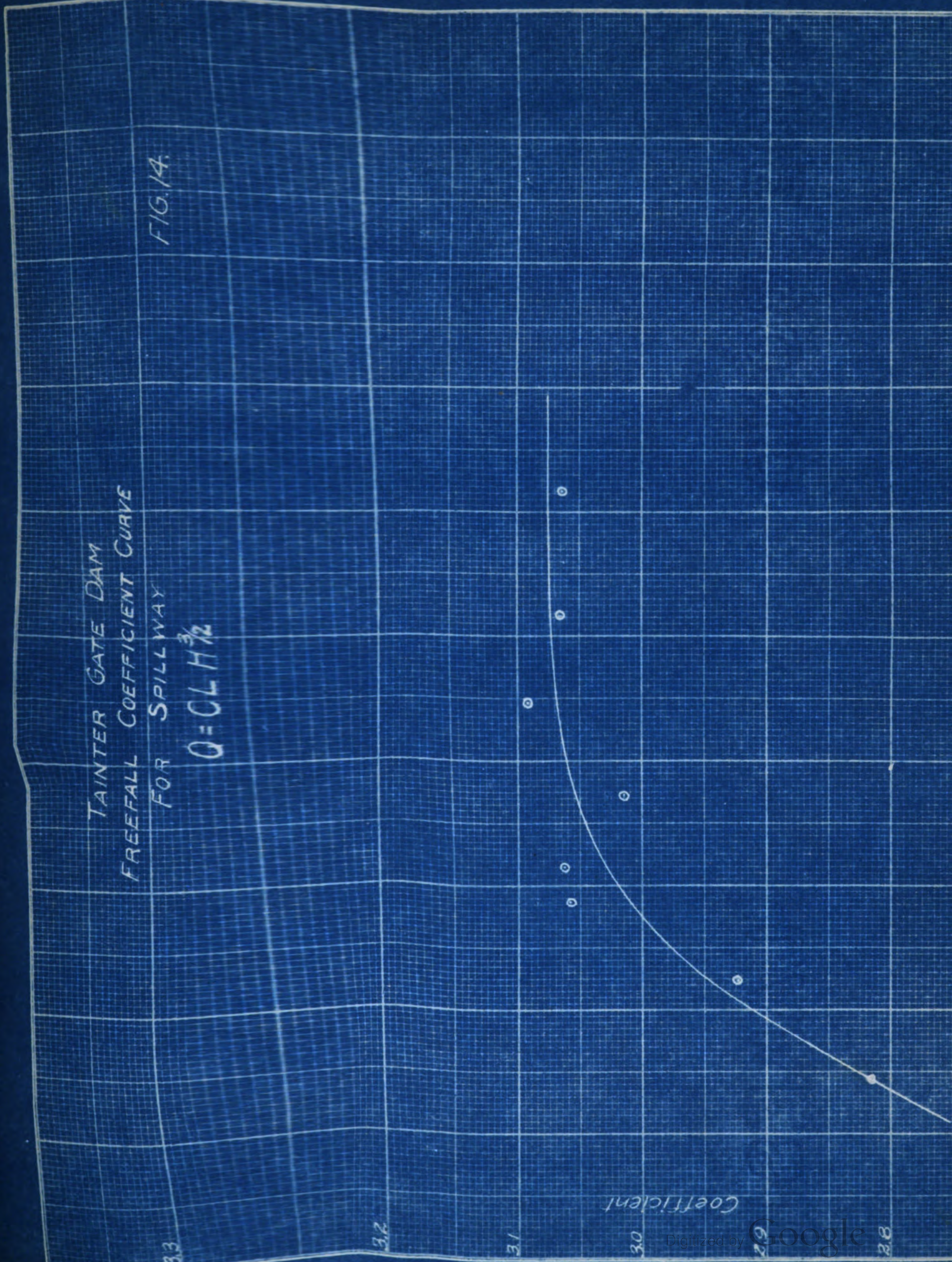

TAINTER GATE DAM
FREEFALL COEFFICIENT CURVE
FOR SPILLWAY
$Q = CLH^{3/2}$
Coefficient
3.3
3.2
3.1
3.0
2.9
2.8

FIG. 14.

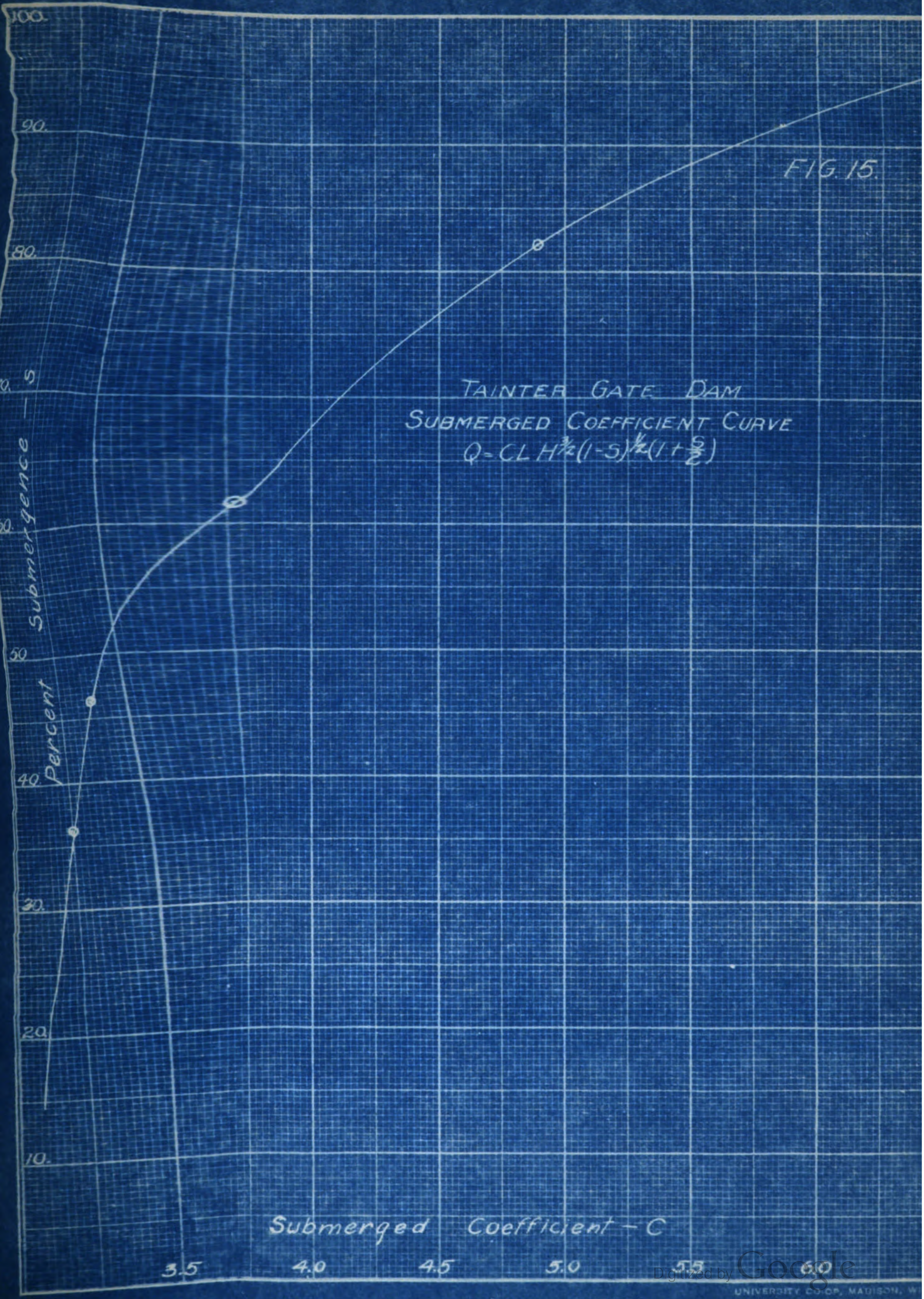
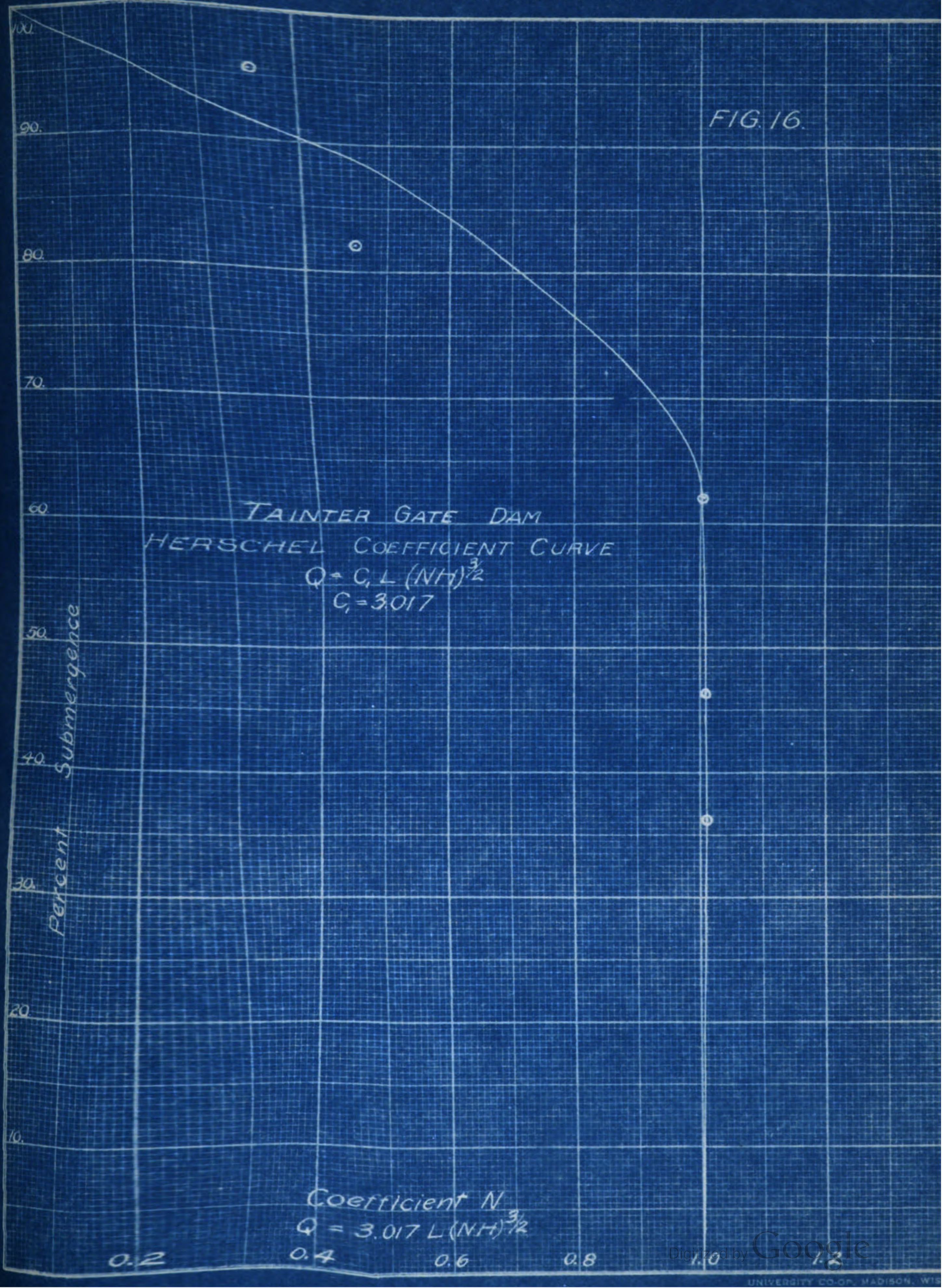


FIG. 15.
TAINTER GATE DAM
SUBMERGED COEFFICIENT CURVE
$Q = CLH^{3/2}(1-S)^{1/2}(1+\frac{S}{2})$
Percent Submergence — S
100.
90.
80.
70.
60.
50.
40.
30.
20.
10.
Submerged Coefficient — C
3.5
4.0
4.5
5.0
5.5
6.0



100.
90.
80.
70.
60.
50.
40.
30.
20.
10.
Percent Submergence
TAINTER GATE DAM
HERSCHEL COEFFICIENT CURVE
$Q = C_1 L (NH)^{3/2}$
$C_1 = 3.017$
Coefficient N
$Q = 3.017\, L\, (NH)^{3/2}$
0.2
0.4
0.6
0.8
1.0
1.2

FIG. 16.

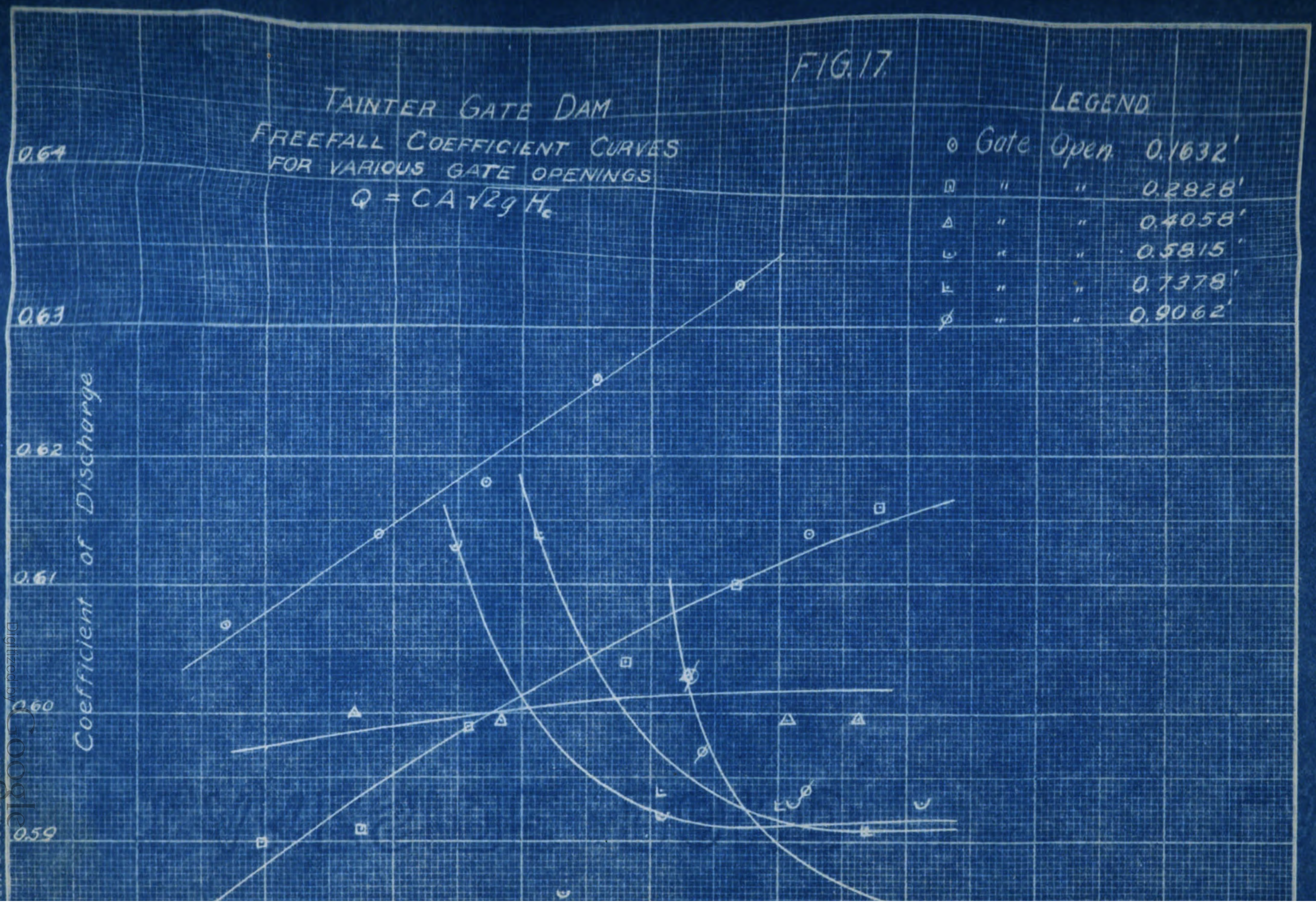


FIG. 17
TAINTER GATE DAM
FREEFALL COEFFICIENT CURVES
FOR VARIOUS GATE OPENINGS
Q = C A √2g Hc
LEGEND
Gate Open 0.1632'
" " 0.2828'
" " 0.4058'
" " 0.5815'
" " 0.7378'
" " 0.9062'
0.64
0.63
0.62
0.61
0.60
0.59
Coefficient of Discharge
FORM C3
COLLEGE OF ENGINEERING UNIVERSITY OF WISCONSIN
UNIVERSITY CO-OP, MAD

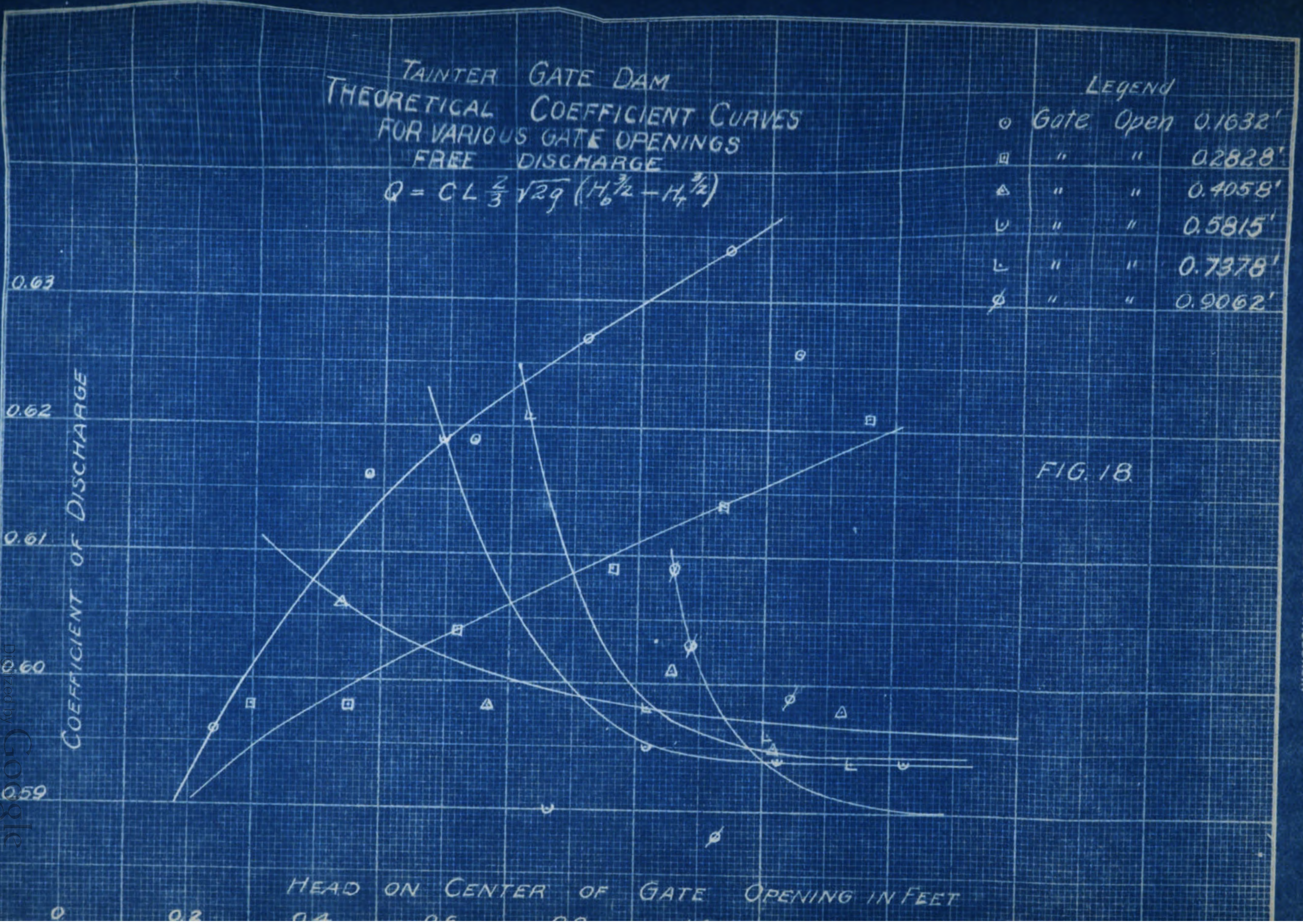
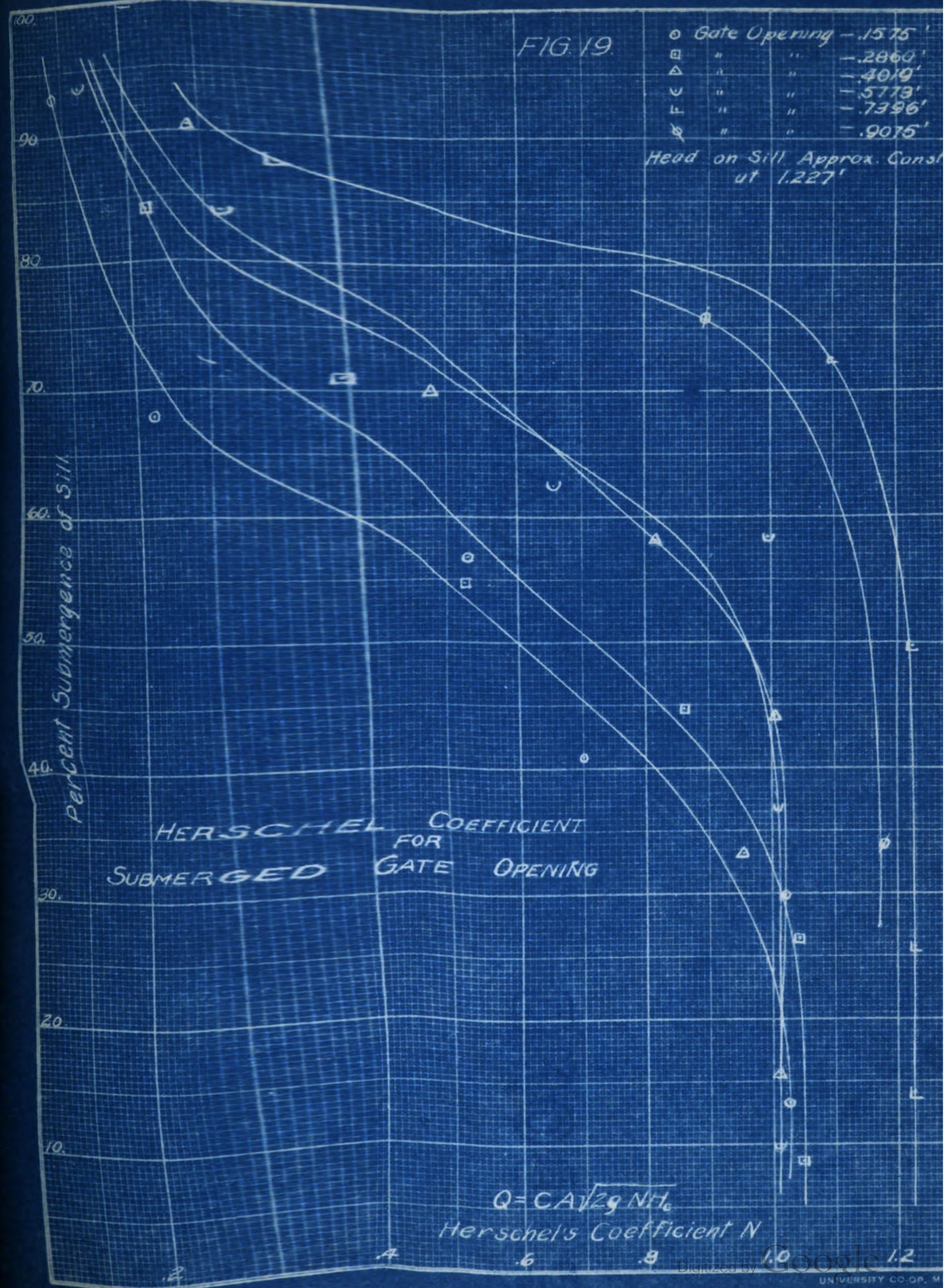

TAINTER GATE DAM
THEORETICAL COEFFICIENT CURVES
FOR VARIOUS GATE OPENINGS
FREE DISCHARGE
$Q = CL\frac{2}{3}\sqrt{2g}\,(H_b^{3/2} - H_t^{3/2})$
LEGEND
Gate Open 0.1632'
" " 0.2828'
" " 0.4058'
" " 0.5815'
" " 0.7378'
" " 0.9062'
COEFFICIENT OF DISCHARGE
0.63
0.62
0.61
0.60
0.59
HEAD ON CENTER OF GATE OPENING IN FEET
0
0.2

FIG. 18

COLLEGE OF ENGINEERING UNIVERSITY OF WISCONSIN



UNIVERSITY CO-OP. M

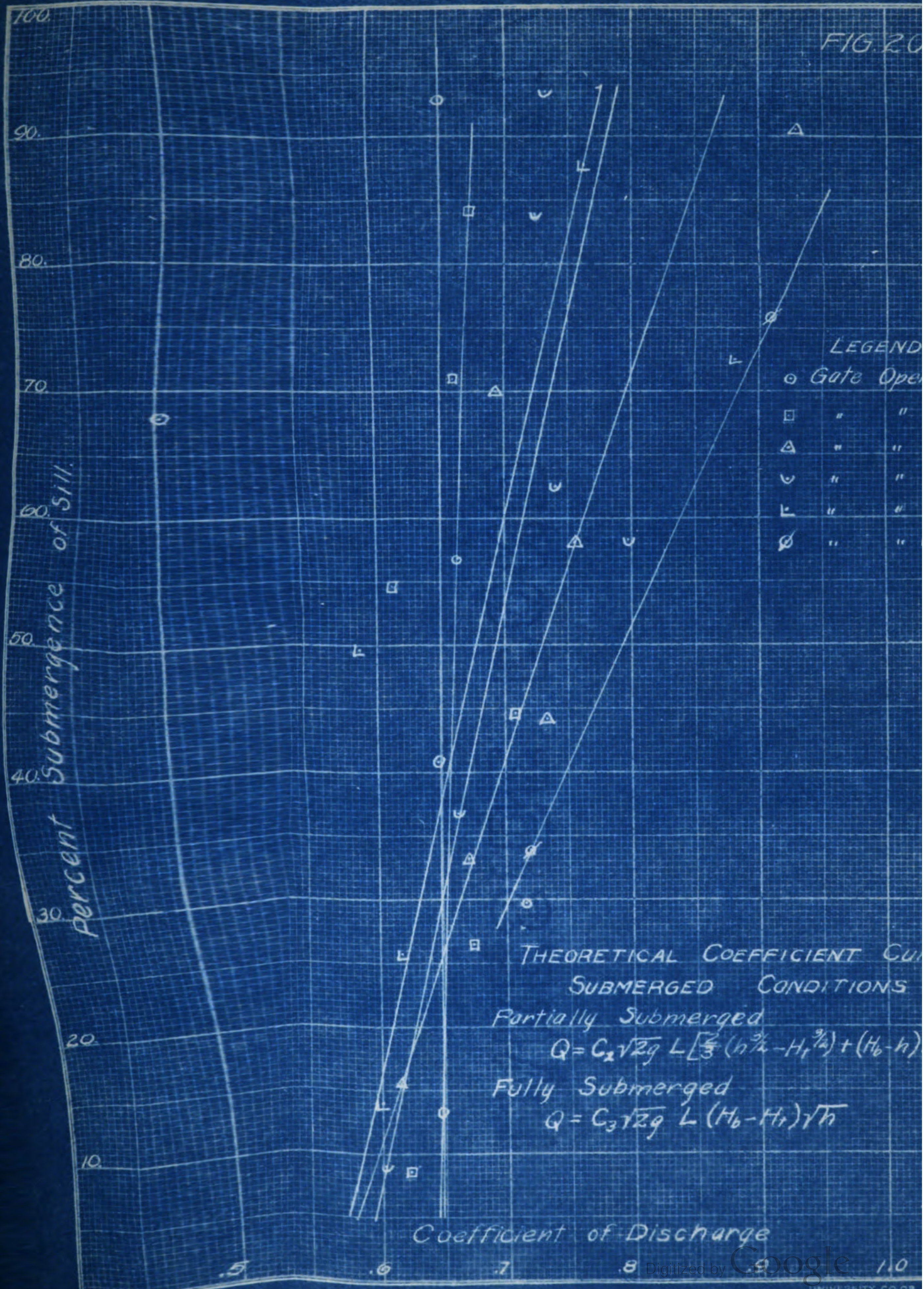
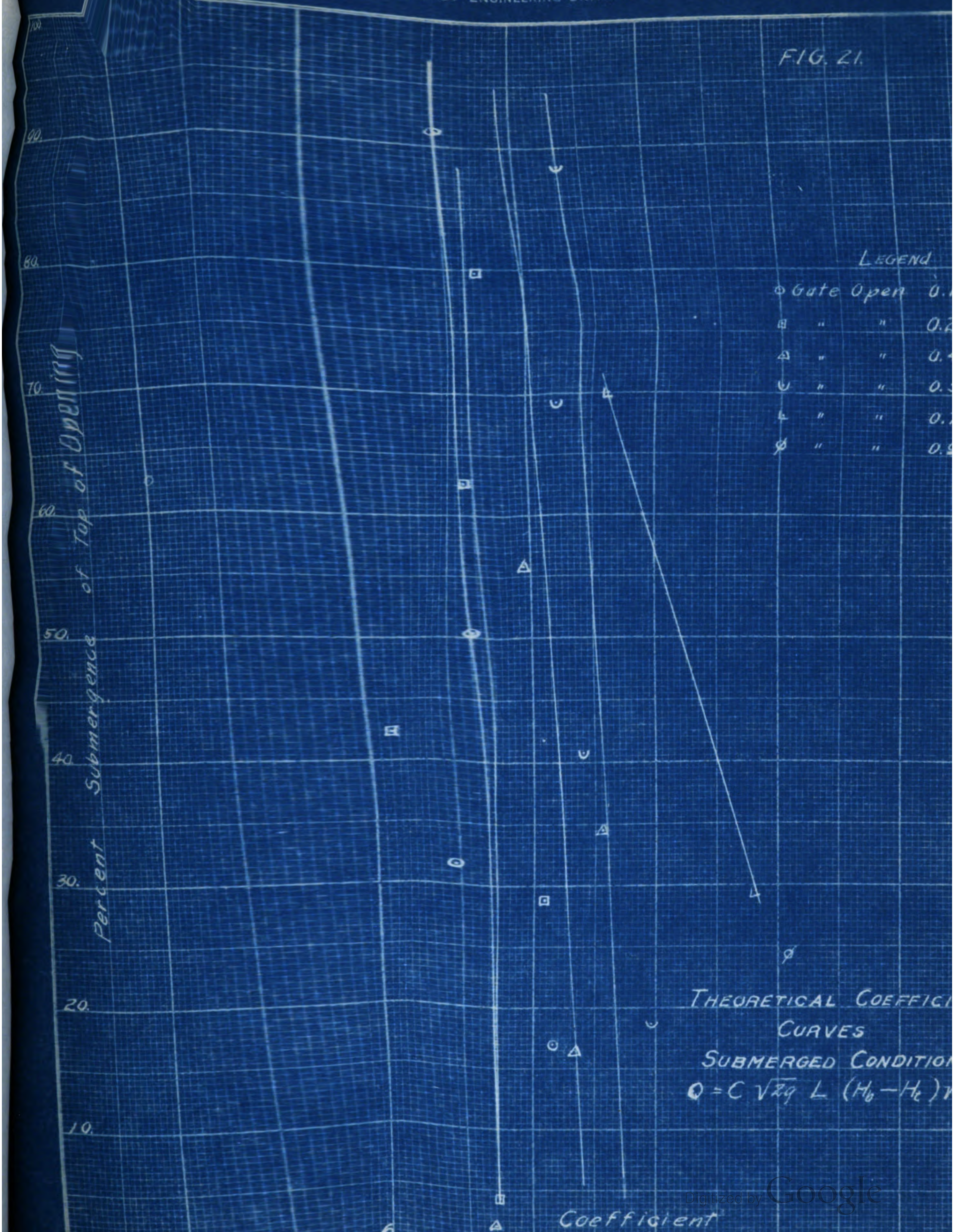


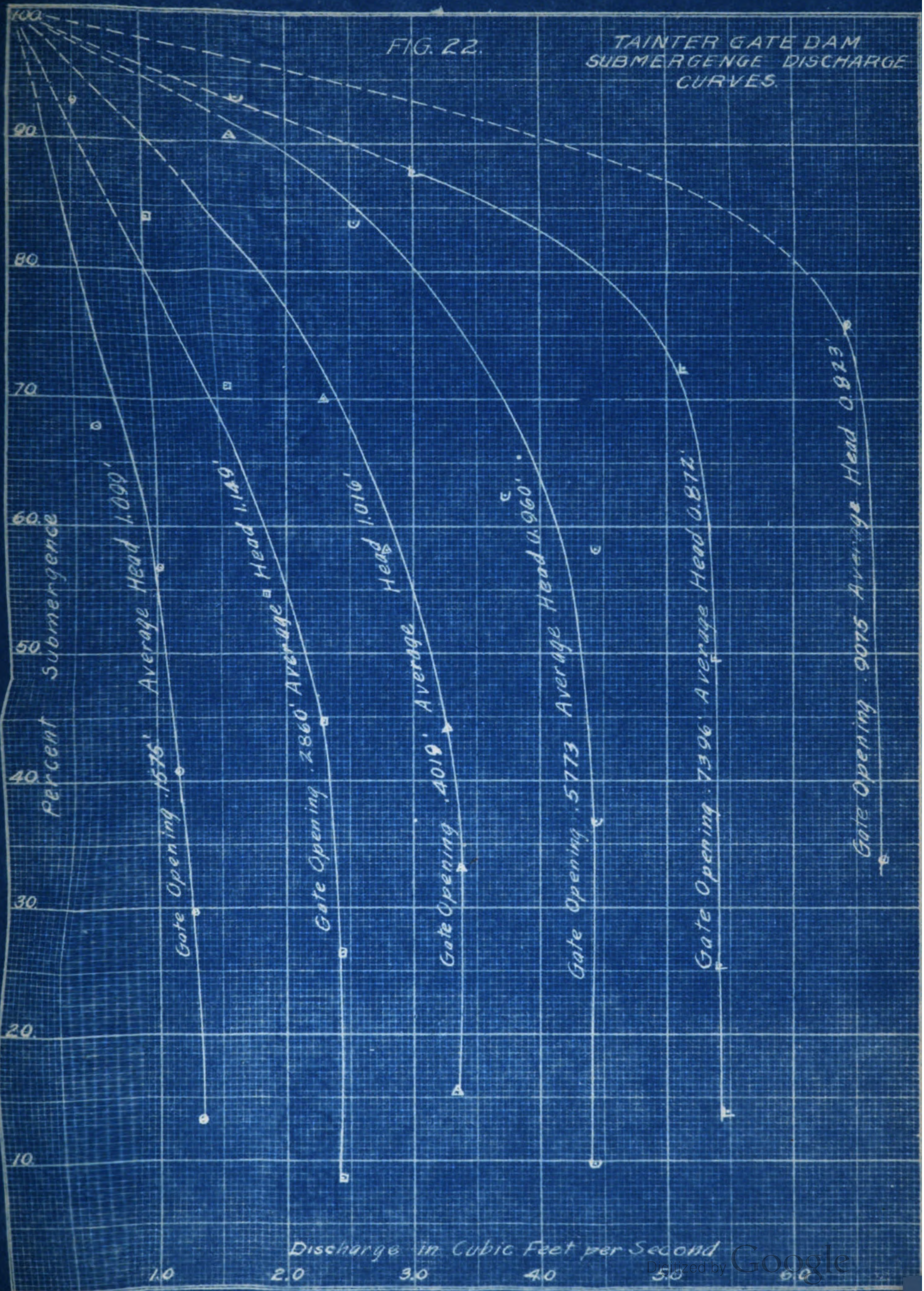
FIG. 2C
LEGEND
Gate Ope
" "
" "
" "
" "
" "
Percent Submergence of Sill.
100.
90.
80.
70.
60.
50.
40.
30.
20.
10.
Coefficient of Discharge
.5
.6
.7
.8
1.0
THEORETICAL COEFFICIENT CU
SUBMERGED CONDITIONS
Partially Submerged
Q=C2√2g L[2/3(h3/2-Ht3/2)+(Hb-h)
Fully Submerged
Q=C3√2g L(Hb-Ht)√h

COLLEGE OF ENGINEERING UNIVERSITY OF WISCONSIN



FORM C2

COLLEGE OF ENGINEERING UNIVERSITY OF WISCONSIN



Discussion of Results

The results obtained for free discharge over the spillway, without the gate in, have been shown by the free-fall coefficient curve, Fig. 14. The values of the coefficient C in the formula $Q = C\,L\,H^{\frac{3}{2}}$ are given, because this equation is in the most convenient form for use, and to make the results comparable to those presented by Horton in Water Supply Paper No. 200. The coefficient is seen to increase very rapidly with the head until the head is slightly greater than the width of the flat weir crest, and apparently approaches a constant value of 3.075 thereafter. This indicates that the effect of the crest is very small when the head is large in comparison with the width of the crest. Data taken by Bazin and the U. S. Geological Survey (see Water Supply Paper No.200) on the flow over dams with 8-inch crests and 2:1 approach and downstream slopes, indicate that the constant coefficient approached at heads of about twice the weir crest width is 3.60. This discrepancy of about 15% the authors believe to be due to the effect of end contractions. The nose of the piers of the experimental dam was shaped as a 90° angle and, while the length of the piers above the crest was sufficient to completely suppress end contractions for small flows, when the flow was large and the drop over the dam considerable, the velocity of the water near the surface was high and there was a considerable contraction due to the blunt pier nose. See Fig. 23.

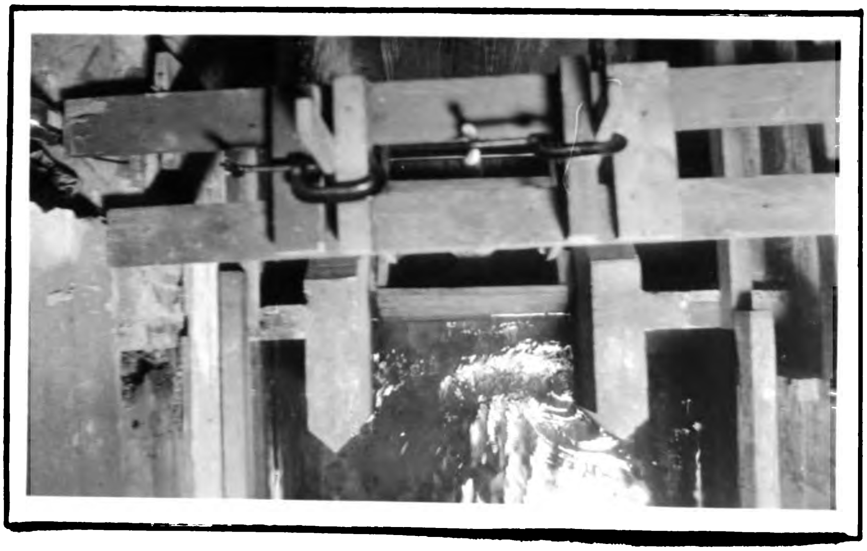


Fig. 23.

Fig. 24.

This effect was very noticeable as it caused a depression of the water surface near the pier faces and a piling up in the center of the spillway. See Fig. 24. This effect was probably increased by the channel being of considerably greater width than the length of the spillway. The water approaching the crest from near the channel walls gained an appreciable velocity and moved at an angle to the weir crest. The authors therefore believe that if two or more gates were used, the coefficient determined by the flow when all were open would be larger than that with only one open.

The submerged coefficient curve given in Fig. 15, shows that the coefficient of discharge of the spillway increases rapidly with increase in submergence. This increase is probably mainly due to the fact that the velocity of flow is decreased with increased submergence, and with lower velocities the effects of crest and end contractions, if any, and of frictional resistance are reduced. Fig. 25 is a view of flow during submerged run #134 showing the overflowing sheet plunging beneath the downstream waters.

Fig.2.5.

The ratio of the head on a free-fall weir to that on a submerged weir for equal discharge, length and crest conditions being the same, is shown graphically in Fig. 16, where N represents that ratio. The effect of drowning is seen to be small until the submergence reaches 60%, while for higher submergences the ratio decreases rapidly.

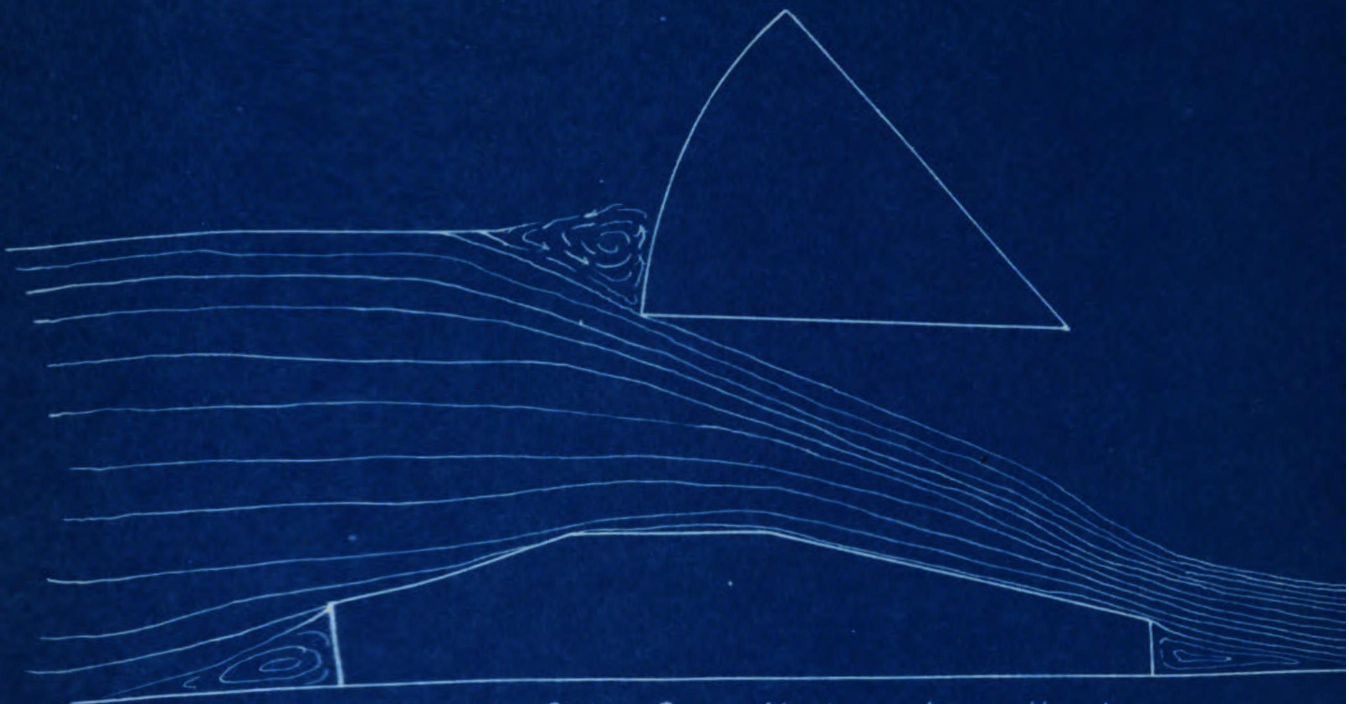
The ordinary discharge coefficients for the apertures made by opening the gate various amounts were computed from

the formula $C = \frac{Q}{A\sqrt{2g\ H_c}}$ and are presented graphically in Fig. 17. The coefficient is seen to increase with the head for small gate openings. This is contrary to the usual experience with sharp edged orifices for which, in general, the coefficient decreases as the head increases. The increase found here is probably due to the fact that the approach slope has the effect of suppressing contraction at the bottom where the head is greatest, and the increase in amount of water carried through this available bottom area due to the increased velocity caused by a higher head is greater than the decrease due to the same change in head caused by the increased contraction at the bottom of the gate where the head and therefore velocity is low. As the size of the gate opening increases, the relative effect of the suppressed and complete contraction have less influence on the total flow and the coefficient ceases to increase with the head and finally even decreases as the head becomes greater.

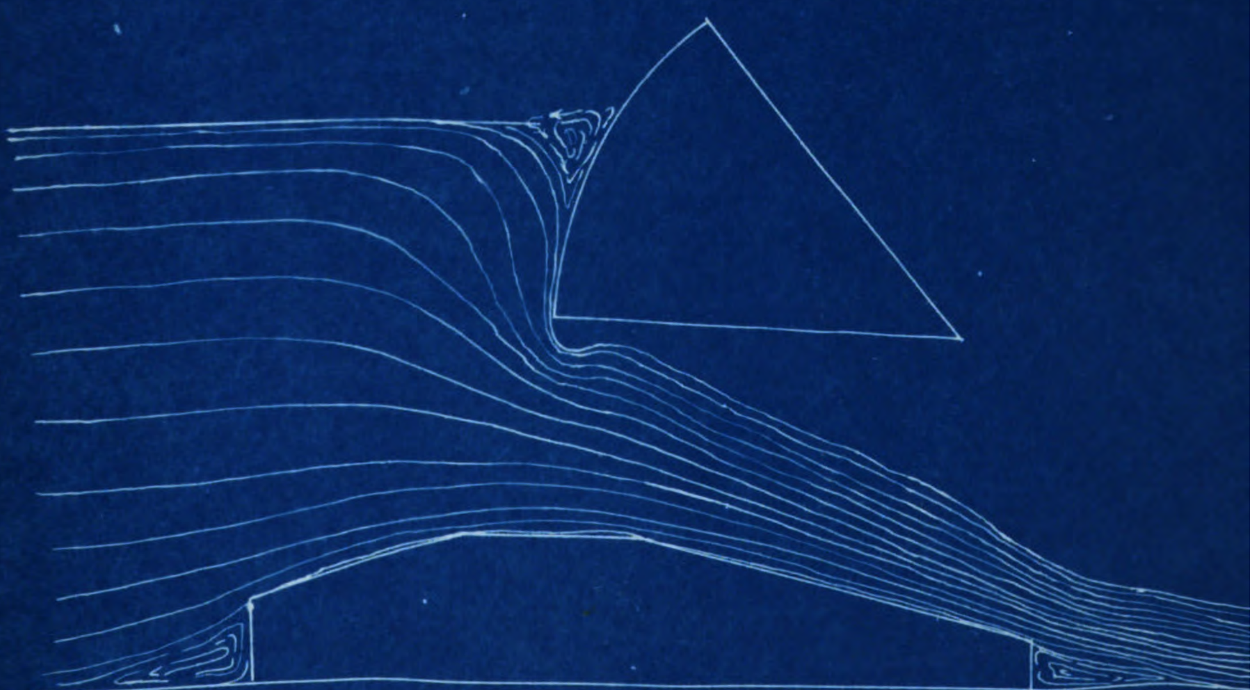
The rapid decrease in the discharge coefficient as the head on the center of large gate openings decreases, may be partially explained by theoretical considerations. Substituting in equation (2), page 17, $H_b = H_c + \frac{W}{2}$ and $H_t = H_c - \frac{W}{2}$, expanding by the binomial theorem, and substituting in equation (2), the result becomes

$$Q = C_1\ L\ W\ \sqrt{2g\ H_c}\ \left[1 - \frac{1}{96}\left(\frac{W}{H_c}\right)^2\ \ldots\ldots\right]$$

in which the series converges rapidly except for relatively large values of $\frac{W}{H_c}$. (Hoskin's, Hydraulics) In using the approximate formula upon which Fig. 17 is based, we have assumed that the head on the center of the aperture is great in comparison with the vertical dimension of the orifice and that all terms except the first in the converging series may be neglected. If however, as in the case of the first run for 0.5815 gate, the ratio $\frac{W}{H_c}$ is nearly equal to one, the second and succeeding terms of the above series become of appreciable value and must be considered. If they are not considered the effect is to give a large value of C, which decreases rapidly until the head increases sufficiently to reduce the ratie $\frac{W}{H_c}$. The effect of neglecting the latter terms of the correct formula for discharge may therefore account for part of the rapid decrease in the value of the coefficient of discharge. However, the consideration mentioned does not explain the entire change, since the variation within the range of these experiments is about 4%, which would require the height of opening to be twice the head on its center, which conditions could not be attained with the opening still acting as an orifice. In these experiments, the largest ratio of height of an opening to the head on its center was about one. Other conditions must therefore be responsible for the decrease in discharging capacity. The general tendency of the coefficient of discharge under considerable heads to decrease with increase



Flow Over Tainter Gate Dam Under Low Head.
FIG. 26



Flow Over Tainter Gate Dam Under High Head.
FIG. 27

in size of opening, is in accordance with the results obtained by other experimenters on large rectangular orifices.

The coefficient of discharge determined from the theoretically correct formula for discharge was also found to decrease with increase in head when the gate opening was large. See Fig. 18. This decrease the authors believe to be due to two principal causes, contraction at the lower edge of the gate and contraction at the piers. As indicated in Fig. 26, when the level of the head water is but little above the lower edge of the gate, the velocity of the upper stratum of water passing through the opening is comparatively low and water drawn down from the surface of the pool approaches the edge of the gate at a small angle with the horizontal. The contraction at the lower edge of the gate is small under those conditions. It may be noted here that in making the experimental runs, it was found necessary to have the upstream water level higher than the lower edge of the gate by from about 0.15 foot for small gate opening to nearly 0.5 foot for a large gate opening before the lower edge of the gate would catch the water surface and act as the upper edge of an orifice. This necessity is due to the increase in velocity of the water as it approaches the crest of the dam and the consequent fall in water surface which begins at a point some distance above the crest. When conditions are as represented in Fig. 27, and the upstream water level is considerably above the lower edge of the gate, the upper strata of water passing through the opening are under a considerable head and hence the velocity is

proportionately large. Under such conditions the water is drawn down to the opening as indicated in Fig. 27, and the contraction at the edge of the gate is large. Such contraction reduces the discharging capacity of the opening and hence reduces the coefficient of discharge.

Mention has been made of the effect of the blunt nosed piers in causing end contractions when the velocity of flow is high, and this effect is probably responsible for part of the reduction in the discharge coefficient when the head on a large gate opening increases.

The theoretical formulas for the discharge through a submerged orifice are quite complicated and coefficients determined for use with them are of little practical value. Therefore, it was thought desirable to consider the discharge from submerged openings in the manner originated by Clemens Herschel for use with submerged weirs. The ratio of the head for free discharge to the head when submerged, for the same orifice discharging water at the same rate, is given by Fig. 19, in which N represents that ratio. The runs for this data were taken with the upstream head as nearly constant as possible, but with several depths of submergence of the sill for each gate opening. The Herschel coefficient curves, Fig. 19, show that for the four small gate openings, and the sill submerged less than 30% the discharge is approximately the same as for a freefall discharge under the same head. For the two larger openings at low submergences the largest dis-

Fig. 28.

Fig. 29.

charge for a given head was secured with the sill submerged. That result can only be explained by considering that the effect of submerging the sill was to reduce the contraction at the piers. The coefficient submergence curves show that for each gate opening there is a certain percent submergence above which the head required to produce any given discharge under submerged conditions becomes rapidly greater than the head for a freefall outlet giving the same discharge. The point at which the change occurs is seen to be very nearly at the percent submergence for which the lower edge of the gate becomes submerged. This change in discharging capacity is probably due to a change in condition of the outflowing sheet of water or nappe. Fig. 29, a view of run #99, shows the outflow conditions when the sill is not submerged but when the channel is obstructed sufficiently to back the water onto the downstream apron. The nappe is seen to fall quickly, and there is no back pressure on the opening. Fig. 30 shows the condition of the nappe for run #102, the gate opening being the same as for run #99, but the downstream level being such that the sill is submerged but the lower edge of the gate is not. The out-flowing sheet has practically the same shape as for the unsubmerged run, the sheet pouring under the downstream water level at the dam and reappearing as violent boiling some distance downstream. The flow through the opening is practically unaffected by such partial submergence. When the channel is restricted until the backwater submerges the entire opening an entirely different

Fig. 30.

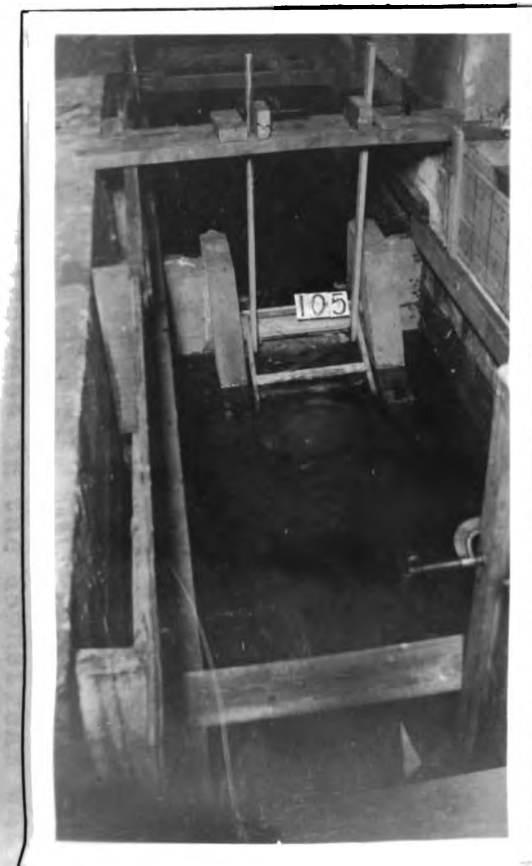


Fig. 31.

condition exists at the gate. (See Fig. 31) The sheet is not then falling as it leaves the opening and its movement is retarded by the back pressure of the downstream water. In this retardation the recoverable portion of the velocity head of the issuing stream is converted into static head, and a series of undulations or waves are formed just below the gate. Part of the velocity head is lost in the shock of the issuing stream rushing into the downstream pool and cannot be recovered as an increase in the downstream water level. That loss causes the decrease in discharging capacity of the completely submerged gate opening.

During some of the low submergence runs the stream below the dam was in a very turbulent and unstable condition. At times the stream emerging from the gate at a high velocity rushed down the center of the channel without enlarging to the full channel width and there was flow back toward the dam on each side. For many of the runs the downstream flow was practically all one one side of the channel as shown in Fig. 28, and there was back flow on the other side. The special still basin used for the gage measuring the downstream head was quite efficient in stilling all disturbances so that a fairly accurate measurement of downstream water level is believed to have been obtained.

Scrutiny of the data and curves for the large gate openings shows that no runs were taken with low submergences, or

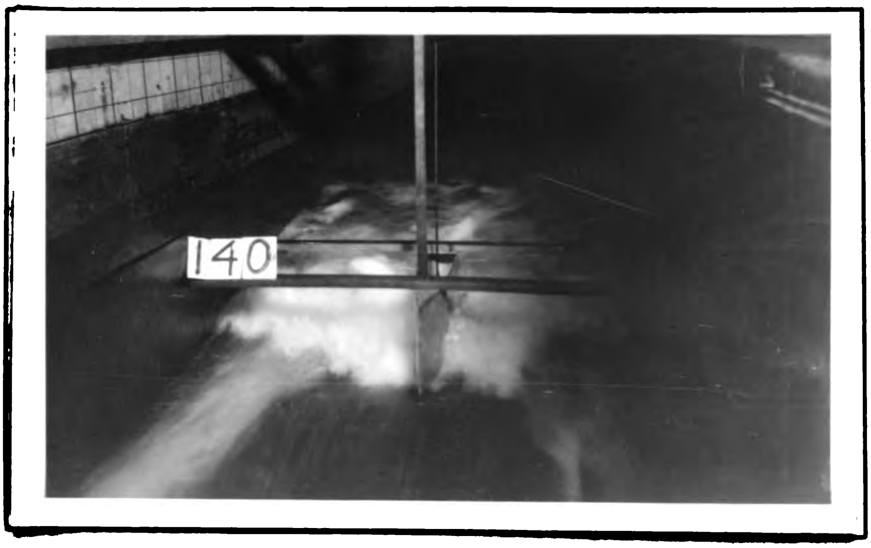


Fig. 32.

Fig. 33.

with the downstream water merely backed upon the apron. These runs could not be taken with the downstream hook-gage in the position eight feet below the dam where it was located for all other runs, for with low submergence the velocity of the water issuing from the large opening was sufficient to cause the standing wave to form below the gage position, as shown in Fig. 32. The gate opening when the views in Figs. 32 and 33 were taken was 0.5773, all conditions except the downstream level being practically as for run #106.

The theoretical coefficient of discharge for the gate opening when both partially and completely submerged has been computed and plotted in Fig. 20 for comparative purposes, although of little practical value. While there is a wide divergence in the values obtained, there is apparently a tendency for the coefficient to increase both with submergence of the sill and with size of gate opening. This increase is probably due partly to the decreased effect of end contractions, as previously discussed, and partly to the effect of eddy currents in reducing the back pressure. These eddy currents were formed by water returning toward the dam along the walls of the channel, circling through the dead water space behind the lower ends of the piers, and flowing down again with the issuing stream. The effect of these streams would not exist if more gates than one were used, but the effect of adjacent streams upon each other would probably be considerable.

The curves shown in Fig. 21 were plotted to show variation in the theoretical discharge coefficient after complete submersion of the gate opening. The coefficient is apparently unchanged by variation in depth of submergence of the upper edge of the opening, but does show a tendency to increase slightly with increase in size of gate opening, due probably to the proportionally smaller influence of contractions and friction.

Conclusions

The coefficient of discharge for the type of spillway experimented with increased at a uniformly decreasing rate until the head is about equal to the width of the flat crest. After that depth of flow is reached the coefficient is practically constant, but is less than that for a weir of the same shape but of infinite length because of incomplete suppression of end contractions.

Submergence has little effect upon the quantity of flow until the downstream head exceeds sixty percent of the upstream head, after which the discharge of the submerged weir is decreased rapidly with increase in submergence.

The coefficient of discharge of the opening left by raising a tainter gate from its sill varies with the ratio of height of opening to head on the center of opening. The value of the coefficient increases with that ratio for small gate openings and decreases with increase in that ratio for large gate openings.

Submergence has little effect upon the discharge through a tainter gate opening until the drowning is complete, after which the head required for a given flow increases with further increase in submergence.

The coefficient of discharge of a tainter gate opening decreases slightly with increase in percent submergence of the top of the opening.

The discharge through a fixed gate opening is influenced little by partial submergence, but decreases with increase in percent submergence after the submergence is complete.

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APPENDIX A

WEIR COEFFICIENT FOR SPILLWAY FREE DISCHARGE

Run No.	Observed head experimental weir D in ft.	$D^{\frac{3}{2}}$	C_1	Q flow per Ft. Experimental weir in cu. ft.sec.	V	$\frac{V^2}{2g}$	H	$H^{\frac{3}{2}}$	C	Head on Weir in Ft.	Total Discharge Cu. Ft. Sec.
35	.189	.0822	2.821	.2327	.166	.0004	.189	.0822	2.831	.1520	.3878
36	.347	.2045	2.931	.6079	.351	.0019	.349	.2062	2.948	.2911	1.0132
37	.469	.3212	3.103	.9948	.500	.0039	.473	.3253	3.058	.4061	1.658
38	.636	.5072	3.065	1.5546	.662	.0068	.643	.5156	3.015	.5495	2.591
39	.784	.6942	3.075	2.1876	.822	.0105	.794	.7075	3.092	.6924	3.646
40	.918	.8795	3.134	2.7588	.935	.0136	.932	.8998	3.066	.8099	4.598
41	1.114	1.1758	3.139	3.6948	1.097	.0187	1.133	1.2060	3.064	.9871	6.158
54	.524			1.180	.569	.0051	.529	.3847	3.063	.4558	1.967

TAINTER GATE DAM
(Without Gate)
CREST SUBMERGED

$$Q = C_1 L (NH)^{\frac{3}{2}}$$

$$C_1 = 3.017$$

Run No.	Observed Head on Measuring Weir	Rate of Flow Cu. Ft./Sec.	Flow per foot Length of Dam	Observed Up-stream Head on Dam	Observed Down-stream Head on Dam	Velocity of Approach	Velocity Head	Corrected Up-stream Head on Dam		Ratio Disch. Submerged to Unsubmerged	Herschel Coefficient	Percent Submergence
			Q	D	d	V	$\frac{V^2}{2g}$	H	$H^{\frac{3}{2}}$	$N^{\frac{3}{2}}$	N	$\frac{d}{H}$
131	1.0313	6.573	3.944	1.164	.430	1.140	.0202	1.184	1.2883	1.015	1.010	.363
132	1.0234	6.497	3.898	1.154	.545	1.228	.0234	1.177	1.2769	1.012	1.008	.463
133	1.0184	6.452	3.871	1.151	.730	1.223	.0232	1.174	1.2721	1.009	1.006	.621
134	1.0096	6.367	3.820	1.174	.983	1.215	.0230	1.197	1.3096	.967	.454	.821
135	0.7402	4.025	2.415	1.113	1.076	0.940	.0137	1.127	1.1964	.669	.355	.955

FREEFALL DISCHARGE

Ordinary Coefficient Series

Run No.	Head on Measuring Weir	Total Discharge Cu. Ft. Sec. Q	Length of Gate L	Height of Gate Opening W	Area of Opening A	Observed Head on Bottom of Opening D_b	Observed Head on Top of Opening D_t	V	$\frac{V^2}{2g}$	Effective Head H_c	$\frac{Q}{A\sqrt{2g\,H_c}}$ C
42	.2116	.6325	1.667'	.1632	.2720	.320	.157	.226	.0008	.239	.607
43	.2735	.9240	"	"	"	.556	.393	.254	.0010	.476	.614
44	.3038	1.080	"	"	"	.721	.558	.256	.0011	.641	.618
45	.3324	1.233	"	"	"	.895	.732	.256	.0011	.815	.626
46	.3635	1.407	"	"	"	1.118	.955	.250	.0010	1.038	.633
47	.3682	1.434	"	"	"	1.227	1.064	.239	.0009	1.146	.614
											Ave..619
48	.3293	1.214	1.667'	.2828	.4715	.435	.154	.379	.0022	.297	.590
49	.3796	1.500	"	"	"	.588	.307	.400	.0025	.450	.591
50	.4265	1.782	"	"	"	.756	.475	.411	.0026	.618	.599
51	.4795	2.119	"	"	"	.998	.717	.408	.0026	.860	.604
52	.5135	2.344	"	"	"	1.171	.890	.403	.0025	1.033	.610
53	.5523	2.612	"	"	"	1.395	1.114	.395	.0025	1.257	.616
											Ave..602

FREEFALL DISCHARGE

Ordinary Coefficient Series

Run No.	Head on Measuring Weir	Total Discharge Cu. Ft. Sec. Q	Length of Gate L	Height of Gate Opening W	Area of Opening A	Observed Heads on Opening Bottom D_b	Observed Heads on Opening Top D_t	V	$\frac{V^2}{2g}$	Effective Head H_c	$\frac{Q}{A\sqrt{2g}\,H_c}$ C
55	.4847	2.153	1.667'	.4058	.6763	.635	.229	.549	.0047	.437	.600
56	.5568	2.643	"	"	"	.867	.461	.557	.0048	.669	.595
57	.6313	3.181	"	"	"	1.152	.746	.555	.0048	.954	.603
58	.6613	3.408	"	"	"	1.315	.909	.540	.0045	1.116	.595
59	.6847	3.587	"	"	"	1.422	1.016	.536	.0045	1.223	.597
										Ave.	.598
60	.6967	3.680	1.667	.5815	.9691	.878	.296	.772	.0093	.596	.613
61	.7349	3.983	"	"	"	1.045	.463	.743	.0086	.763	.586
62	.7879	4.415	"	"	"	1.200	.618	.747	.0086	.918	.592
63	.8433	4.882	"	"	"	1.404	.822	.736	.0084	1.121	.593
64	.8923	5.305	"	"	"	1.604	1.022	.724	.0082	1.321	.593
										Ave.	.595
65	.8764	5.167	1.667	.7378	1.230	1.082	.344	.937	.0136	.727	.614
66	.9269	5.613	"	"	"	1.274	.536	.910	.0129	.918	.594
67	.9866	6.155	"	"	"	1.462	.724	.900	.0126	1.106	.593
68	1.0235	6.498	"	"	"	1.597	.859	.888	.0122	1.240	.591
										Ave.	.598

Run No.	Head of Measuring Weir	V	$\frac{V^2}{2g}$	Effective Head H_c	$\frac{Q}{A\sqrt{2g\,H_c}} = C$
129	.991[illegible]	1.084	.0181	.959	.603
130	.982[illegible]	1.045	.0170	1.028	.583
137	.987[illegible]	1.060	.0175	1.141	.594
138	.927[illegible]	1.070	.0178	.981	.597

APPENDIX B

Calibration of Measuring Weir

The calibration of the two foot measuring weir was one of the problems to be met in the course of the experiment. This weir had been calibrated several times before, but it was thought best to run a check calibration inasmuch as some of the conditions might not be the same as before.

Apparatus

The Reservoir

The Laboratory Reservoir, the source of water supply for this calibration, is located on a bluff adjacent to the laboratory. It is a circular concrete type of construction, fifteen feet in depth and fifty feet in diameter, providing a storage capacity of approximately thirty thousand cubic feet of water. The elevation of the filled reservoir is sixty-five feet above the level of Lake Mendota.

An observation pit is located on the side of the reservoir towards the laboratory. In this pit are placed water column gages by means of which the depth of the water in the reservoir can be read directly to the hundredth part of a foot. By the use of a vernier attachment it was possible to read to a thousandth of a foot. As this reservoir had previously been accurately calibrated, it served as a simple

means of measuring the amount of water passing over the weir.

The Pipe Line

A ten inch, 12 inches inside laboratory, pipe line led from the reservoir to the weir box. This line had a control valve a few feet from the weir box.

The Signals

Inasmuch as there could be no direct communication between the observers at the gages of the reservoir, and at the weir box, it was necessary to employ some system for indicating when readings should be taken. This was effected by the electrical signal system which had been previously installed. The operator at the weir box had control of a knife switch by means of which he could signal to the operator in the reservoir pit.

Method

Observers were stationed in the reservoir pit, and at the weir box hook gage. The observer at the weir box hook gage also controlled the flow of water by means of the control valve.

The operator at the weir box opened the supply valve until the desired discharge was obtained. When the conditions of flow had steadied down, he signalled the observer in the reservoir pit to start reading. At the end of five minutes

the operator at the weir signalled the observed in the reservoir pit to stop reading. During the run readings were taken at half minute intervals. This process was carried out with the head on the weir ranging from one-tenth to one foot. It was then repeated, this time starting with a one foot head, and decreasing by decrements of approximately one-tenth of a foot.

Results

The computations of the results was a simple matter. There being available a large size head-excess calibration curve similar to that shown in Fig. 35., and the difference in gage readings at the reservoir being known the total discharge was calculated. The discharge in second feet was secured by dividing by the length of run.

The average head on the weir was obtained by averaging the data taken by the observer at the weir box. A logarithmic plotting of head against discharge, as shown in Fig. 34., was then made. A table, accompanying this appendix, was made giving the logarithm of the head on the measuring weir, and the corresponding logarithm of the discharge. This was used in the computations.

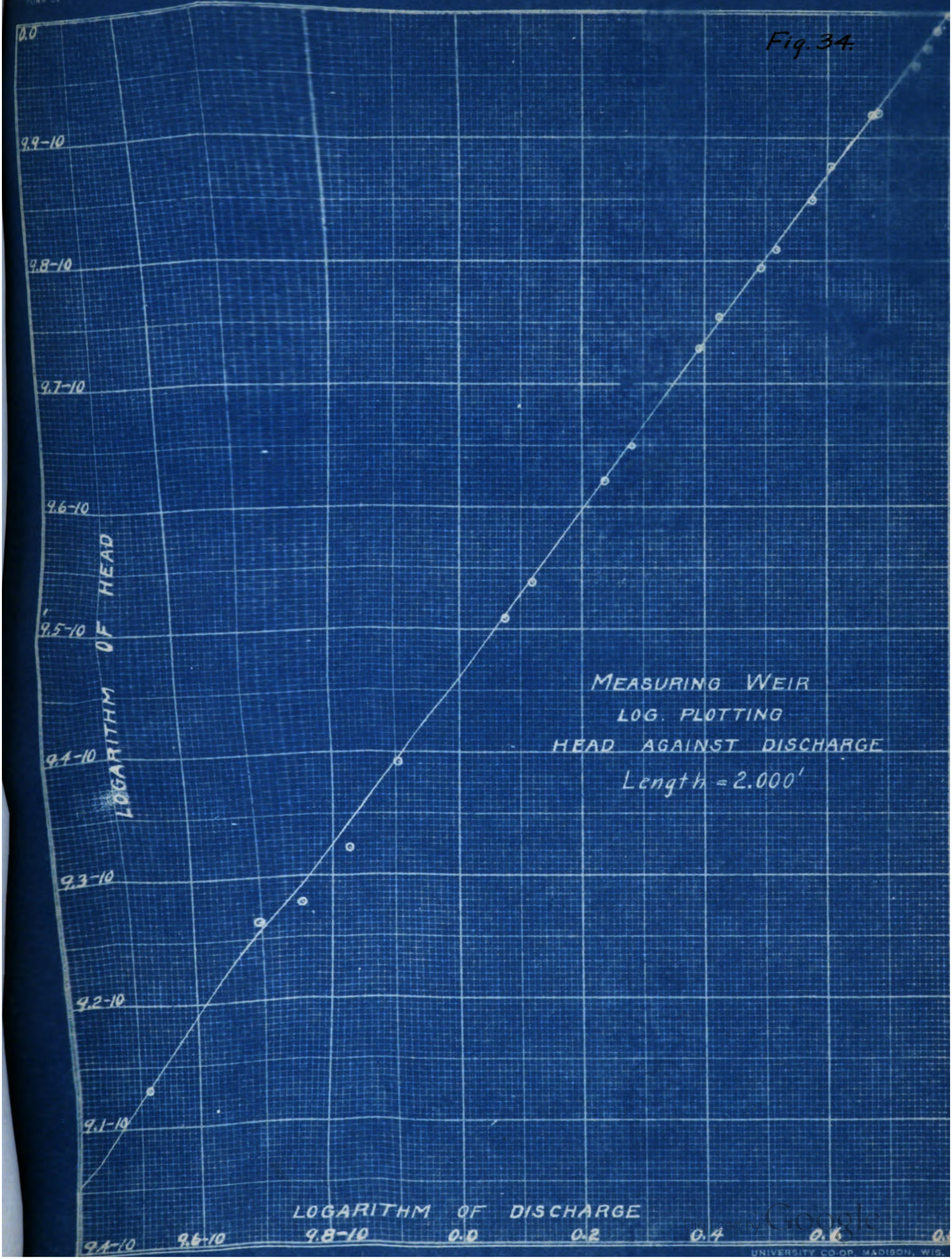

MEASURING WEIR
LOG. PLOTTING
HEAD AGAINST DISCHARGE
Length = 2.000′
LOGARITHM OF HEAD
LOGARITHM OF DISCHARGE
0.0
9.9-10
9.8-10
9.7-10
9.6-10
9.5-10
9.4-10
9.3-10
9.2-10
9.1-10
9.4-10
9.6-10
9.8-10
0.0
0.2
0.4
0.6

Fig. 34.

Run No.	...ge ...on ...Ft.	Logar-ithm of Head on Weir
2	834	9.2634
3	231	9.5093
4	171	9.6202
5	364	9.7295
6	232	9.7946
7	074	9.8497
8	279	9.9180
9	390	9.9727
10	712	9.9873
22	899	9.2785
23	452	9.5380
24	465	9.6498
25	686	9.7548
26	449	9.8095
27	532	9.8769
28	318	9.9200
29	072	9.9577
30	986	9.9994
31	845	8.9268
32	352	9.1310
33	101	9.3224
34	464	9.3916

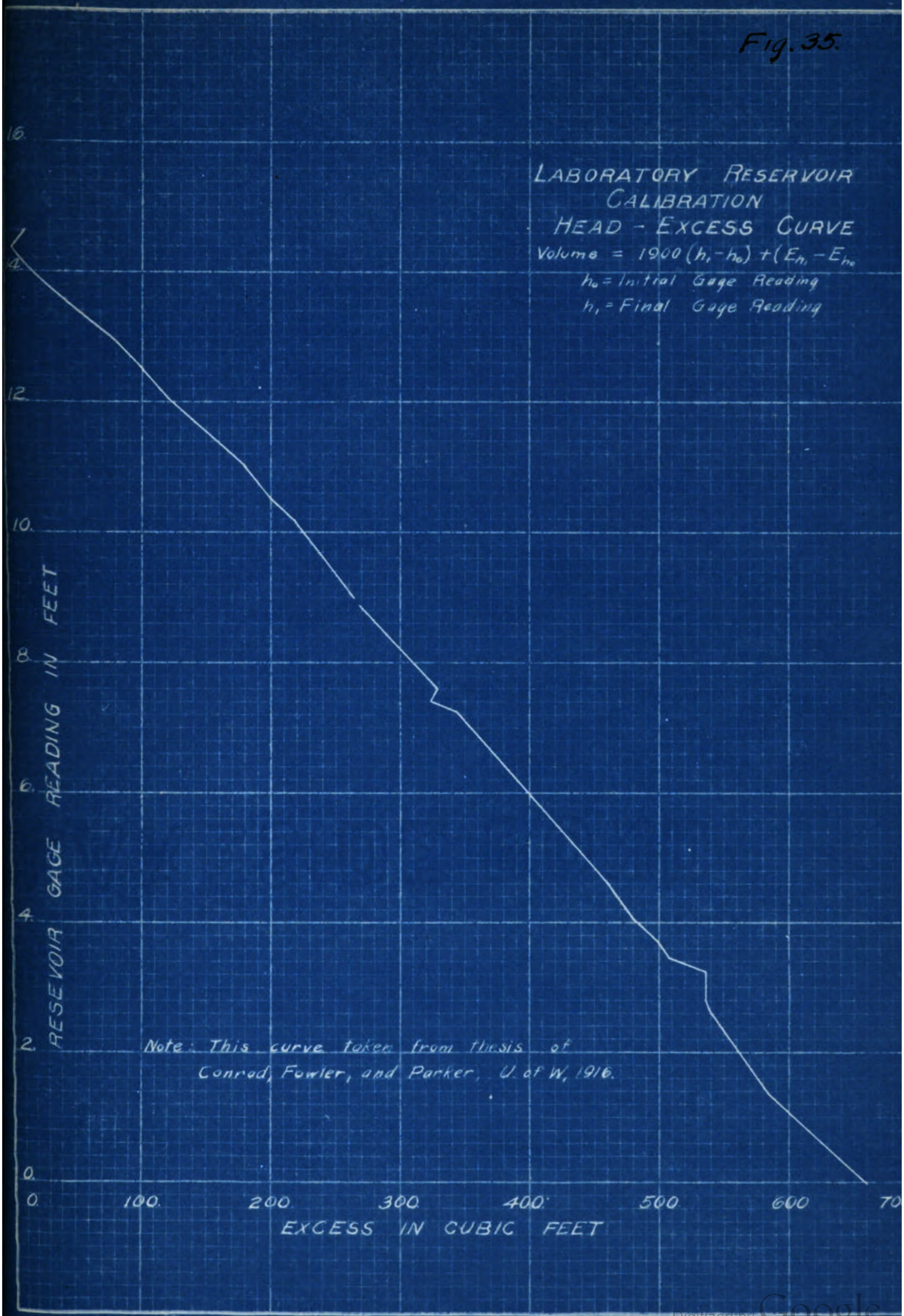
Log. Head	Log. Disch.	Head	Disch.	Head	Disch.	Head	Disch.	Head
8.900	9.1722	50	9.2461	9.000	9.3200	9.050	9.3938	9.100
.901	.1737	1	.2476	1	.3215	1	.3953	1
.902	.1752	2	.2491	2	.3230	2	.3968	2
.903	.1767	3	.2506	3	.3245	3	.3983	3
.904	.1782	4	.2521	4	.3260	4	.3998	4
.905	.1796	5	.2535	5	.3274	5	.4012	5
.906	.1811	6	.2550	6	.3289	6	.4027	6
.907	.1826	7	.2565	7	.3304	7	.4042	7
.908	.1841	8	.2580	8	.3319	8	.4057	8
.909	.1856	9	.2595	9	.3333	9	.4072	9
8.910	.1870	.960	.2609	9.010	.3347	9.060	.4086	9.110
.911	.1885	1	.2624	1	.3362	1	.4101	1
.912	.1900	2	.2639	2	.3377	2	.4116	2
.913	.1915	3	.2654	3	.3392	3	.4131	3
.914	.1930	4	.2669	4	.3407	4	.4146	4
.915	.1944	5	.2683	5	.3421	5	.4160	5
.916	.1959	6	.2698	6	.3436	6	.4175	6
.917	.1974	7	.2713	7	.3451	7	.4190	7
.918	.1989	8	.2728	8	.3466	8	.4205	8
.919	.200[illegible]	9	.2743	9	.3471	9	.4220	9
8.920	.201[illegible]	8.970	.2757	9.020	.3495	9.070	.4234	9.120
.921	.203[illegible]	1	.2772	1	.3510	1	.4249	1
.922	.204[illegible]	2	.2787	2	.3525	2	.4264	2
.923	.206[illegible]	3	.2802	3	.3540	3	.4279	3
.924	.20[illegible]	4	.2816	4	.3555	4	.4294	4
.925	.20[illegible]	5	.2830	5	.3569	5	.4308	5
.926	.2107	6	.2845	6	.3584	6	.4323	6
.927	.2122	7	.2860	7	.3549	7	.4338	7
.928	.2[illegible]37	8	.2875	8	.3614	8	.4353	8
.929	.2[illegible]51	9	.2890	9	.3629	9	.4368	9
8.930	.2[illegible]65	8.980	.2904	9.030	.3642	9.080	.4381	9.130
.931	.2180	1	.2919	1	.3657	1	.4396	1
.932	.2195	2	.2934	2	.3672	2	.4411	2
.933	.2210	3	.2949	3	.3687	3	.4426	3
.934	.2225	4	.2964	4	.3702	4	.4441	4
.935	.2239	5	.2978	5	.3716	5	.4455	5
.936	.2254	6	.2993	6	.3731	6	.4470	6
.937	.2269	7	.3008	7	.3746	7	.4485	7
.938	.2284	8	.3023	8	.3761	8	.4500	8
.939	.2299	9	.3038	9	.3776	9	.4515	9
8.940	.2313	8.990	.3052	9.040	.3790	9.090	.4529	9.140
.941	.2328	1	.3067	1	.3805	1	.4544	1
.942	.2343	2	.3082	2	.3820	2	.4559	2
.943	.2358	3	.3097	3	.3835	3	.4574	3
.944	.2373	4	.3112	4	.3850	4	.4589	4
.945	.2387	5	.3126	5	.3864	5	.4603	5
.946	.2402	6	.3141	6	.3879	6	.4618	6
.947	.2417	7	.3156	7	.3894	7	.4633	7
.948	.2432	8	.3171	8	.3909	8	.4648	8
.949	.2447	9	.3186	9	.3924	9	.4663	9

Log. Head	Log. Disch.	Log. Head	Log. Disch.	Log. Head	Log. Disch.	Log. Head	Log. Disch.	Log. Head	Log. Disch.	Log. Head	Log. Disch.	Log. Head	Log. Disch.
9.250	9.6894	9.300	9.7633	9.350	9.8372	9.400	9.9111	9.450	9.9850	9.500	0.0589	9.550	0.1328
1	.6909	1	.7648	1	.8387	1	.9126	1	.9865	1	.0604	1	.1343
2	.6924	2	.7663	2	.8402	2	.9141	2	.9880	2	.0619	2	.1358
3	.6939	3	.7678	3	.8417	3	.9156	3	.9895	3	.0634	3	.1373
4	.6954	4	.7693	4	.8432	4	.9171	4	.9910	4	.0649	4	.1388
5	.6968	5	.7707	5	.8446	5	.9185	5	.9924	5	.0663	5	.1402
6	.6983	6	.7722	6	.8461	6	.9200	6	.9939	6	.0678	6	.1417
7	.6998	7	.7737	7	.8476	7	.9215	7	.9954	7	.0693	7	.1432
8	.7013	8	.7752	8	.8491	8	.9230	8	.9969	8	.0708	8	.1447
9	.7028	9	.7767	9	.8506	9	.9245	9	.9984	9	.0723	9	.1462
9.260	.7042	9.310	.7781	9.360	.8520	9.410	.9259	9.460	.9998	9.510	.0737	9.560	.1476
1	.7057	1	.7796	1	.8535	1	.9274	1	0.0013	1	.0752	1	.1491
2	.7072	2	.7811	2	.8550	2	.9289	2	.0028	2	.0767	2	.1506
3	.7087	3	.7826	3	.8565	3	.9304	3	.0043	3	.0782	3	.1521
4	.7102	4	.7841	4	.8580	4	.9319	4	.0058	4	.0797	4	.1536
5	.7116	5	.7855	5	.8594	5	.9333	5	0.0072	5	.0811	5	.1550
6	.7131	6	.7870	6	.8609	6	.9348	6	.0087	6	.0826	6	.1565
7	.7146	7	.7885	7	.8624	7	.9363	7	.0102	7	.0841	7	.1580
8	.7161	8	.7900	8	.8639	8	.9378	8	.0117	8	.0856	8	.1595
9	.7176	9	.7915	9	.8654	9	.9393	9	.0132	9	.0871	9	.1610
9.270	.7190	9.320	.7929	9.370	.8668	9.420	.9407	9.470	.0146	9.520	.0885	9.570	.1624
1	.7205	1	.7944	1	.8683	1	.9422	1	.0161	1	.0900	1	.1639
2	.7220	2	.7959	2	.8698	2	.9437	2	.0176	2	.0915	2	.1654
3	.7235	3	.7974	3	.8713	3	.9452	3	.0191	3	.0930	3	.1669
4	.7250	4	.7989	4	.8728	4	.9467	4	.0206	4	.0945	4	.1684
5	.7264	5	.8003	5	.8742	5	.9481	5	.0220	5	.0959	5	.1698
6	.7279	6	.8018	6	.8757	6	.9496	6	.0235	6	.0974	6	.1713
7	.7294	7	.8033	7	.8772	7	.9511	7	.0250	7	.0989	7	.1728
8	.7309	8	.8048	8	.8787	8	.9526	8	.0265	8	.1004	8	.1743
9	.7323	9	.8062	9	.8801	9	.9540	9	.0279	9	.1018	9	.1757
9.280	.7337	9.330	.8076	9.380	.8815	9.430	.9554	9.480	.0293	9.530	.1032	9.580	.1771
1	.7352	1	.8091	1	.8830	1	.9569	1	.0308	1	.1047	1	.1786
2	.7367	2	.8106	2	.8845	2	.9584	2	.0323	2	.1062	2	.1801
3	.7382	3	.8121	3	.8860	3	.9599	3	.0338	3	.1077	3	.1816
4	.7397	4	.8136	4	.8875	4	.9614	4	.0353	4	.1092	4	.1831
5	.7411	5	.8150	5	.8889	5	.9628	5	.0367	5	.1106	5	.1845
6	.7426	6	.8165	6	.8904	6	.9643	6	.0382	6	.1121	6	.1860
7	.7441	7	.8180	7	.8919	7	.9658	7	.0397	7	.1136	7	.1875
8	.7456	8	.8195	8	.8934	8	.9673	8	.0412	8	.1151	8	.1890
9	.7471	9	.8210	9	.8949	9	.9688	9	.0427	9	.1166	9	.1905
9.290	.7485	9.340	.8224	9.390	.8963	9.440	.9702	9.490	.0441	9.540	.1180	9.590	.1919
1	.7500	1	.8239	1	.8978	1	.9717	1	.0456	1	.1195	1	.1934
2	.7515	2	.8254	2	.8993	2	.9732	2	.0471	2	.1210	2	.1949
3	.7530	3	.8269	3	.9008	3	.9747	3	.0486	3	.1225	3	.1964
4	.7545	4	.8284	4	.9023	4	.9762	4	.0501	4	.1240	4	.1979
5	.7559	5	.8298	5	.9037	5	.9776	5	.0515	5	.1254	5	.1993
6	.7574	6	.8313	6	.9052	6	.9791	6	.0530	6	.1269	6	.2008
7	.7589	7	.8328	7	.9067	7	.9806	7	.0545	7	.1284	7	.2023
8	.7604	8	.8343	8	.9082	8	.9821	8	.0560	8	.1299	8	.2038
9	.7619	9	.8358	9	.9097	9	.9836	9	.0575	9	.1314	9	.2053

Log. Head	Log. Disch.	Log. Head	Log. Disch.	Log. Head	Log. Disch.	Log. Head	Log. Disch.	Log. Head	Log. Disch.	Log. Head	Log. Disch.
9.600	0.2067	9.650	0.2806	9.700	0.3545	9.750	0.4284	9.800	0.5023	9.850	0.5762
1	.2082	1	.2821	1	.3560	1	.4299	1	.5038	1	.5777
2	.2097	2	.2836	2	.3575	2	.4314	2	.5053	2	.5792
3	.2112	3	.2851	3	.3590	3	.4329	3	.5068	3	.5807
4	.2127	4	.2866	4	.3605	4	.4344	4	.5083	4	.5822
5	.2141	5	.2880	5	.3619	5	.4358	5	.5097	5	.5836
6	.2156	6	.2895	6	.3634	6	.4373	6	.5112	6	.5851
7	.2171	7	.2910	7	.3649	7	.4388	7	.5127	7	.5866
8	.2186	8	.2925	8	.3664	8	.4403	8	.5142	8	.5881
9	.2201	9	.2940	9	.3679	9	.4418	9	.5157	9	.5896
9.610	.2215	9.660	.2954	9.710	.3693	9.760	.4432	9.810	.5171	9.860	.5910
1	.2230	1	.2969	1	.3708	1	.4447	1	.5186	1	.5925
2	.2245	2	.2984	2	.3723	2	.4462	2	.5201	2	.5940
3	.2260	3	.2999	3	.3738	3	.4477	3	.5216	3	.5955
4	.2275	4	.3014	4	.3753	4	.4492	4	.5231	4	.5970
5	.2289	5	.3028	5	.3767	5	.4506	5	.5245	5	.5984
6	.2304	6	.3043	6	.3782	6	.4521	6	.5260	6	[illegible]
7	.2319	7	.3058	7	.3797	7	.4536	7	.5275	7	[illegible]
8	.2334	8	.3073	8	.3812	8	.4551	8	.5290	8	[illegible]
9	.2349	9	.3088	9	.3827	9	.4566	9	.5305	9	[illegible]
9.620	.2363	9.670	.3102	9.720	.3841	9.770	.4580	9.820	.5319	9.870	[illegible]
1	.2378	1	.3117	1	.3856	1	.4595	1	.5334	1	.6073
2	.2393	2	.3132	2	.3871	2	.4610	2	.5349	2	.6088
3	.2408	3	.3147	3	.3886	3	.4625	3	.5364	3	.6103
4	.2423	4	.3162	4	.3901	4	.4640	4	.5379	4	.6118
5	.2437	5	.3176	5	.3915	5	.4654	5	.5393	5	.6132
6	.2452	6	.3191	6	.3930	6	.4669	6	.5408	6	.6147
7	.2467	7	.3206	7	.3945	7	.4684	7	.5423	7	.6162
8	.2482	8	.3221	8	.3960	8	.4699	8	.5438	8	.6176
9	.2496	9	.3236	9	.3974	9	.4713	9	.5452	9	.6191
9.630	.2510	9.680	.3250	9.730	.3988	9.780	.4727	9.830	.5466	9.880	.6206
1	.2525	1	.3265	1	.4003	1	.4742	1	.5481	1	.6221
2	.2540	2	.3280	2	.4018	2	.4757	2	.5496	2	.6236
3	.2555	3	.3295	3	.4033	3	.4772	3	.5511	3	.6251
4	.2570	4	.3309	4	.4048	4	.4787	4	.5526	4	.6265
5	.2584	5	.3323	5	.4062	5	.4801	5	.5540	5	.6279
6	.2599	6	.3338	6	.4077	6	.4816	6	.5555	6	.6294
7	.2614	7	.3353	7	.4092	7	.4831	7	.5570	7	.6309
8	.2629	8	.3368	8	.4107	8	.4846	8	.5585	8	.6324
9	.2644	9	.3383	9	.4122	9	.4861	9	.5600	9	.6339
9.640	.2658	9.690	.3397	9.740	.4136	9.790	.4875	9.840	.5614	9.890	.6353
1	.2673	1	.3412	1	.4151	1	.4890	1	.5629	1	.6368
2	.2688	2	.3427	2	.4166	2	.4905	2	.5644	2	.6383
3	.2703	3	.3442	3	.4181	3	.4920	3	.5659	3	.6398
4	.2718	4	.3457	4	.4196	4	.4935	4	.5674	4	.6413
5	.2732	5	.3471	5	.4210	5	.4949	5	.5688	5	.6427
6	.2747	6	.3486	6	.4225	6	.4964	6	.5703	6	.6442
7	.2762	7	.3501	7	.4240	7	.4979	7	.5718	7	.6457
8	.2777	8	.3516	8	.4255	8	.4994	8	.5733	8	.6472
9	.2792	9	.3531	9	.4270	9	.5009	9	.5748	9	.6487

Log. Head	Log. Disch.	Log. Head	Log. Disch	Log. Head	Log. Disch.	Log. Head	Log. Disch	Log. Head	Log. Disch	Log. Head	Log. Disch.	Log. Head	Log. Disch
9.950	0.7240	9.970	0.7536	9.990	0.7831	0.010	0.8127	0.030	0.8422	0.050	0.8718	0.070	0.9014
1	.7255	1	.7551	1	.7846	1	.8142	1	.8437	1	.8733	1	.9029
2	.7270	2	.7566	2	.7861	2	.8157	2	.8452	2	.8748	2	.9044
3	.7285	3	.7571	3	.7876	3	.8172	3	.8467	3	.8763	3	.9059
4	.7300	4	.7586	4	.7891	4	.8187	4	.8482	4	.8778	4	.9074
5	.7314	5	.7610	5	.7905	5	.8201	5	.8496	5	.8792	5	.9088
6	.7329	6	.7625	6	.7920	6	.8216	6	.8511	6	.8807	6	.9103
7	.7344	7	.7640	7	.7935	7	.8231	7	.8526	7	.8822	7	.9118
8	.7359	8	.7655	8	.7950	8	.8246	8	.8541	8	.8837	8	.9133
9	.7374	9	.7670	9	.7965	9	.8261	9	.8556	9	.8852	9	.9148
9.960	.7388	9.980	.7684	0.000	.7979	0.020	.8275	0.040	.8570	0.060	.8866	0.080	.9161
1	.7403	1	.7699	1	.7994	1	.8290	1	.8585	1	.8881	1	.9176
2	.7418	2	.7714	2	.8009	2	.8305	2	.8600	2	.8896	2	.9191
3	.7433	3	.7729	3	.8024	3	.8320	3	.8615	3	.8911	3	.9207
4	.7448	4	.7743	4	.8039	4	.8335	4	.8630	4	.8926	4	.9221
5	.7462	5	.7757	5	.8053	5	.8349	5	.8644	5	.8940	5	.9235
6	.7477	6	.7772	6	.8068	6	.8364	6	.8659	6	.8955	6	.9250
7	.7492	7	.7787	7	.8083	7	.8374	7	.8674	7	.8970	7	.9265
8	.7507	8	.7802	8	.8098	8	.8394	8	.8689	8	.8985	8	.9280
9	.7522	9	.7817	9	.8113	9	.8408	9	.8704	9	.9000	9	.9295

Fig. 35.


LABORATORY RESERVOIR
CALIBRATION
HEAD - EXCESS CURVE
Volume = 1900 (h, - h) + (E - E
h = Initial Gage Reading
h, = Final Gage Reading
Note: This curve taken from thesis of
Conrad, Fowler, and Parker, U. of W, 1916.
RESEVOIR GAGE READING IN FEET
EXCESS IN CUBIC FEET
0 100 200 300 400 500 600 70
0. 2. 4. 6. 8. 10. 12. 14. 16.

APPENDIX C

Calibration of the Venturi Meter

The calibration of the venturi meter was necessary before it could be used for measuring the flow of water. This apparatus had been calibrated previously, but the set up differed and it was necessary to calibrate it again.

Apparatus

The apparatus used in the calibration of this instrument was the same as that used for the weir and will not be described here. The electric signalling system was again used advantageously.

Method

The method employed was the same as that used in calibrating the weir, observers being stationed at the mercury pressure gage of the venturi meter, and in the observation pit of the reservoir. Calibrations were made with increasing and decreasing heads.

Results

The results were computed in the same manner as for the weir and a logarithmic plotting of difference in head against discharge was made as shown in Fig. 36.

Run No.	D a o i	Venturi Meter Reading Ft. of Hg. Inlet	Throat
69		1.533	2.052
70		1.308	2.280
71		1.031	2.550
119		1.540	1.961
120		1.262	2.236
121		1.020	2.477
122		0.733	2.764
123		0.468	3.034
124		0.442	3.062
125		0.733	2.764
126		1.012	2.491
127		1.266	2.243
128		1.542	1.968

FORM C2

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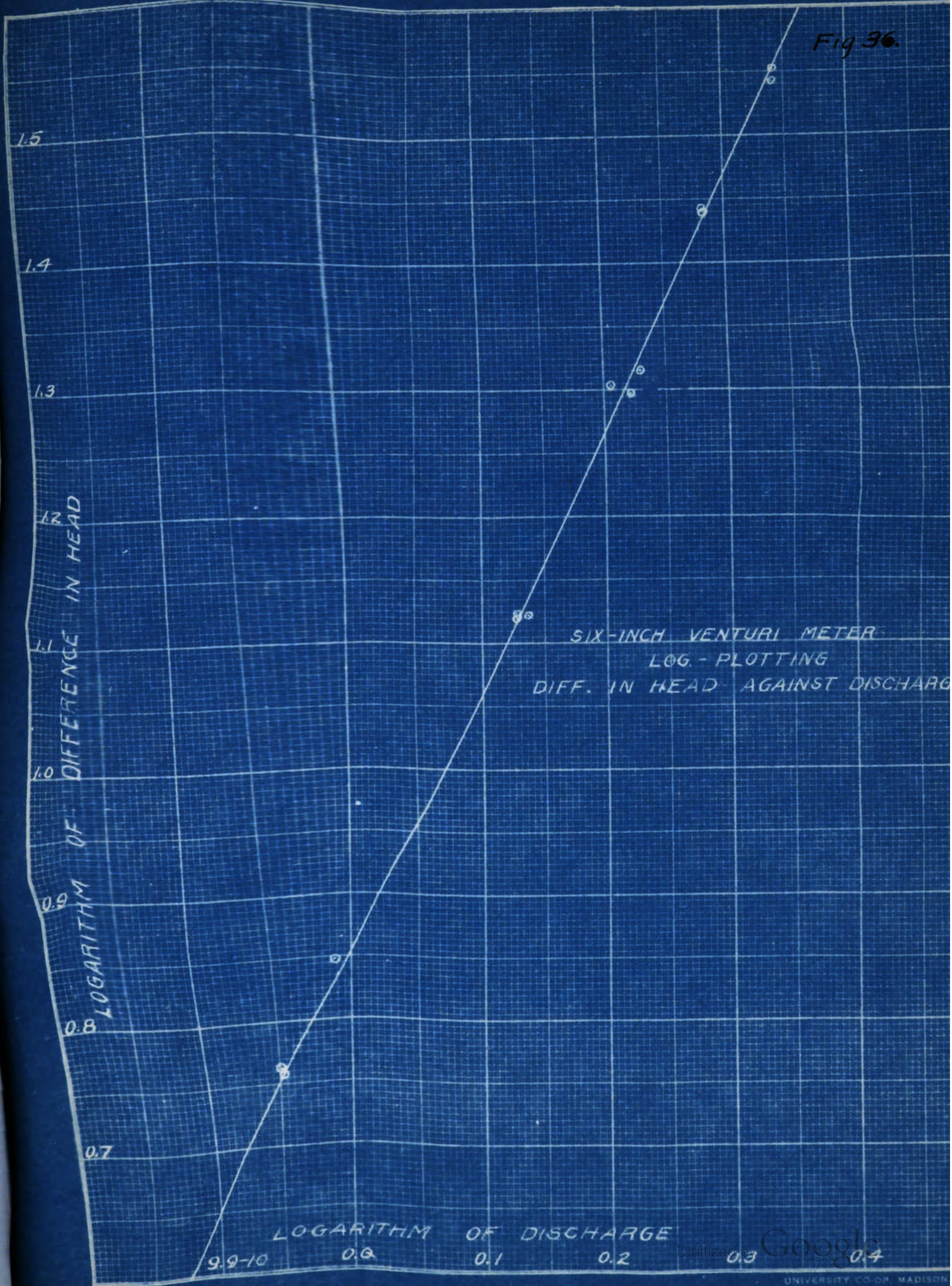
Fig 36.



APPROVED

The foregoing thesis is hereby approved as a creditable study of an engineering subject, carried out and presented in a manner sufficiently satisfactory to warrant its acceptance as a prerequisite to the degree for which it has been submitted. It is to be understood that by this approval, the undersigned does not necessarily endorse or approve any statement made, opinions expressed or conclusions drawn therein, but approves the thesis only for the purpose for which it is submitted.

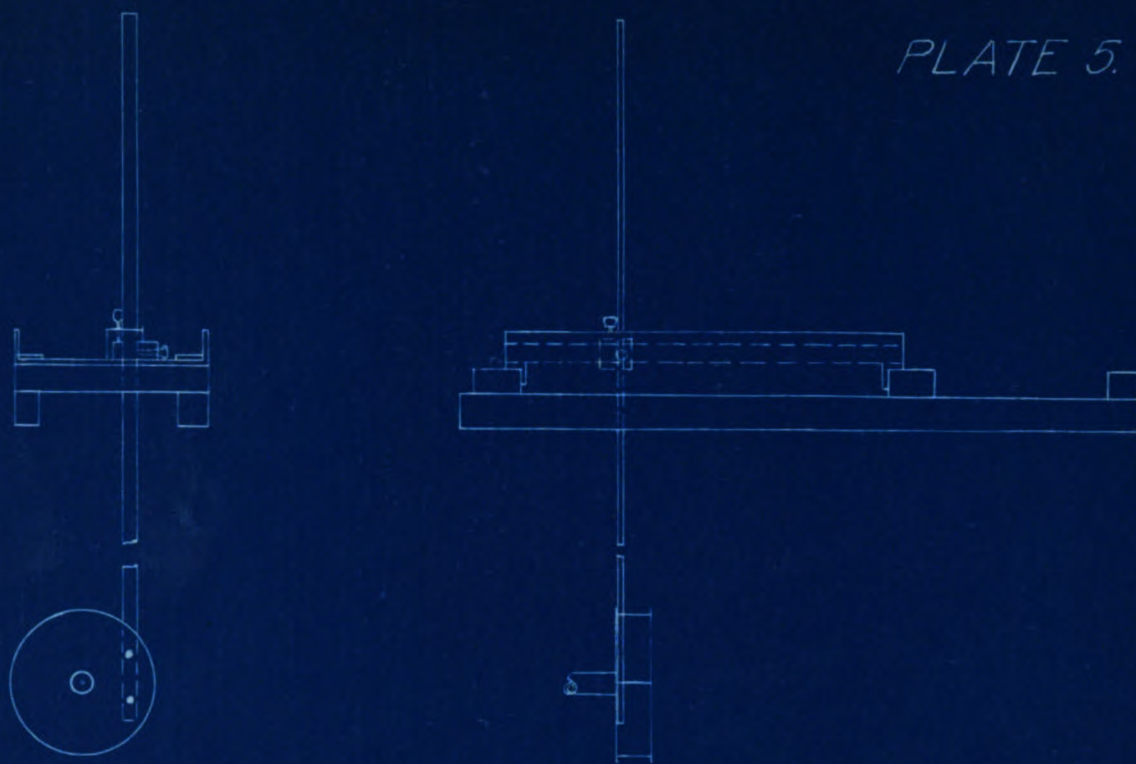
(Name) Chas. D. Corp

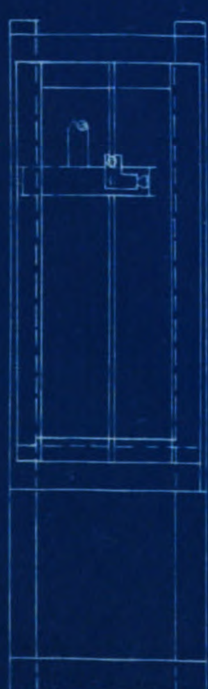
(Title) Prof. of Hydraulic & Sanitary Engineering

Plate I

AN OF APPARATUS
N DETERMINING
THRU TAINTER GATE
ONSIN HYDRAULICS LABRATORY
SCALE $\frac{1}{4}$" = 1'

PLATE 5.





PIEZOMETER
USED IN EXPERIMENTS ON
DISCHARGE THROUGH TAINTER GATES
Scale 1" = 1'